



EM 1110-2-2400
2 June 03

**US Army Corps
of Engineers®**

ENGINEERING AND DESIGN

Structural Design and Evaluation of Outlet Works

ENGINEER MANUAL

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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-2-2400

CECW-EW

Manual
No. 1110-2-2400

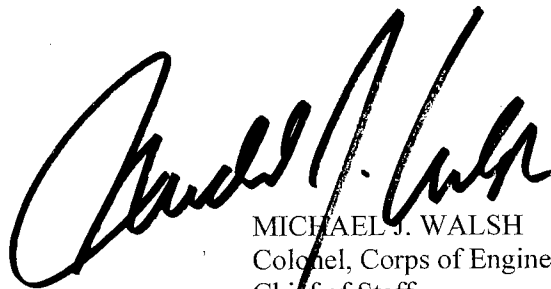
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Engineering and Design
STRUCTURAL DESIGN AND EVALUATION OF OUTLET WORKS

- 1. Purpose.** This manual provides guidance for the planning and structural design and analysis of intake structures and other outlet works features used on U.S. Army Corps of Engineers projects for the purpose of flood control, water supply, water quality and temperature control, recreation, or hydropower.
- 2. Applicability.** This manual applies to all HQUSACE elements and USACE commands having responsibilities for the design of civil works projects.
- 3. Distribution Statement.** This publication is approved for public release; distribution is unlimited.
- 4. Scope of the Manual.** This manual presents guidance for the planning and design of outlet works structures, with special emphasis on intake towers. Other outlet works structures covered include tunnels, cut-and-cover conduits, access bridges, gate structures, and approach and discharge channel structures. Appurtenant features covered include trashracks, gates, valves, and mechanical and electrical operating equipment. Chapter 2 presents general planning and design information; Chapters 3 and 4 provide structural and seismic design guidance; Chapter 5 describes trashracks, bulkheads, gates, valves, and operating equipment; and Chapter 6 covers access bridge design requirements and describes special detailing considerations. Appendices B through E cover the seismic design and evaluation of intake towers.

FOR THE COMMANDER:

5 Appendices
(See Table of Contents)



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Chief of Staff

This manual supersedes EM 1110-2-2400, 2 November 1964.

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Chapter 1

Introduction

1-1. Purpose

This manual provides guidance for the planning and structural design and analysis of intake structures and other outlet works features used on U.S. Army Corps of Engineers projects for the purpose of flood control, water supply, water quality and temperature control, recreation, or hydropower.

1-2. Applicability

This manual applies to all HQUSACE elements and USACE commands having responsibilities for the design of civil works projects.

1-3. Distribution

This publication is approved for public release; distribution is unlimited.

1-4. References

Required and related publications are listed in Appendix A.

1-5. Scope

a. Overview. This manual presents guidance for the planning and design of outlet works structures, with special emphasis on intake towers. Other outlet works structures covered include tunnels, cut-and-cover conduits, access bridges, gate structures, and approach and discharge channel structures. Appurtenant features covered include trashracks, gates, valves, and mechanical and electrical operating equipment. Chapter 2 presents general planning and design information; Chapters 3 and 4 provide structural and seismic design guidance; Chapter 5 describes trashracks, bulkheads, gates, valves, and operating equipment; and Chapter 6 covers access bridge design requirements and describes special detailing considerations. Appendices B through E cover the seismic design and evaluation of intake towers.

b. Guidance limitation.

(1) The procedures in this manual are intended for outlet works structures founded on rock. Outlet works structures should not be founded on soil unless piling is provided to support the structure in the event the soil supporting the structure is eroded. The design of pile-supported structures is covered in EM 1110-2-2906.

(2) Intake structures contained within or attached to concrete gravity dams are not addressed in this manual. Details of intake structures that are integral with a concrete gravity dam can be found in the U.S. Bureau of Reclamation handbook, "Design of Gravity Dams" (U.S. Bureau of Reclamation 1976).

Chapter 2 General Design Considerations

2-1. General

This chapter presents structural design considerations for outlet works structures used with embankment dams, or concrete dams with outlet works detached from the dam. A detached outlet works may be the most economical for a concrete dam when the dam is located in a narrow canyon with restricted space for the outlet features. The hydraulic design of outlet works is covered in EM 1110-2-1602. Intakes through concrete dams, normally called sluices, are covered in EM 1110-2-2200 and EM 1110-2-1602.

a. Outlet works. Outlet works consist of a combination of structures designed to control the release of water from the reservoir as required for project purposes or operation. The components of outlet works, starting from the upstream end, typically consist of an approach channel, an intake structure, a conduit or a tunnel, a control gate chamber (located in the intake structure, within the conduit, or at the downstream end of the conduit), an exit chute, an energy dissipater, and a discharge channel. Outlet works are frequently used to pass diversion flows during construction, regulate flood flows, aid in emptying the reservoir in an emergency condition, and permit reservoir lowering for inspections and special repairs. Typical arrangements of outlet works are shown in Figures 2-1, 2-2, and 2-3. The sizing of the outlet works should take into account the possibility of using it to reduce the size or frequency of spillway discharges. The necessity of emergency drawdown capability and low flow discharge capability should be considered during the outlet works planning phase. The selection of type and arrangement of outlet works structures should be based upon consideration of the costs of operation and maintenance likely to be incurred during the project life. Reliability under emergency flood conditions is a fundamental operational requirement of outlet works facilities.

b. Intake structures. The intake structure may serve several different functions in the outlet works system. Besides forming the entrance, it may include (1) a trashrack to block debris, (2) fish entrances, (3) multilevel ports or weirs for water temperature control, (4) temporary diversion openings, (5) water supply and irrigation intakes, (6) bulkhead or stoplogs for closure, and (7) control gates and devices.

c. Outlet tunnels or conduits. Water passage through or around the dam is provided through tunnels in rock abutments or through cut-and-cover conduits through the base of an embankment type dam.

(1) A tunnel in a rock abutment is preferred when it is more economical than other means and when topographic conditions permit its use. Narrow rock abutments, permitting a relatively short tunnel, are conditions preferable for tunnels. The selection of the number and size of tunnels depends on hydraulic requirements, the maximum size of tunnel for the rock conditions encountered, and overall economics. When a tunnel is used also for diversion, its sizing is often controlled by the diversion releases required during construction. Figure 2-1 shows a tunnel outlet works.

(2) Cut-and-cover conduits are economical for low- or moderate-head embankment dams because of lower construction costs and a shorter alignment. A typical cut-and-cover conduit is placed near streambed elevation, preferably outside the normal river channel to facilitate construction. It should be placed within the most competent portion of the foundation and, when practicable, on suitable rock to keep potential settlement to a minimum. High-head dams will require a thick concrete section to resist high embankment loads. Figure 2-2 shows a cut-and-cover outlet works.

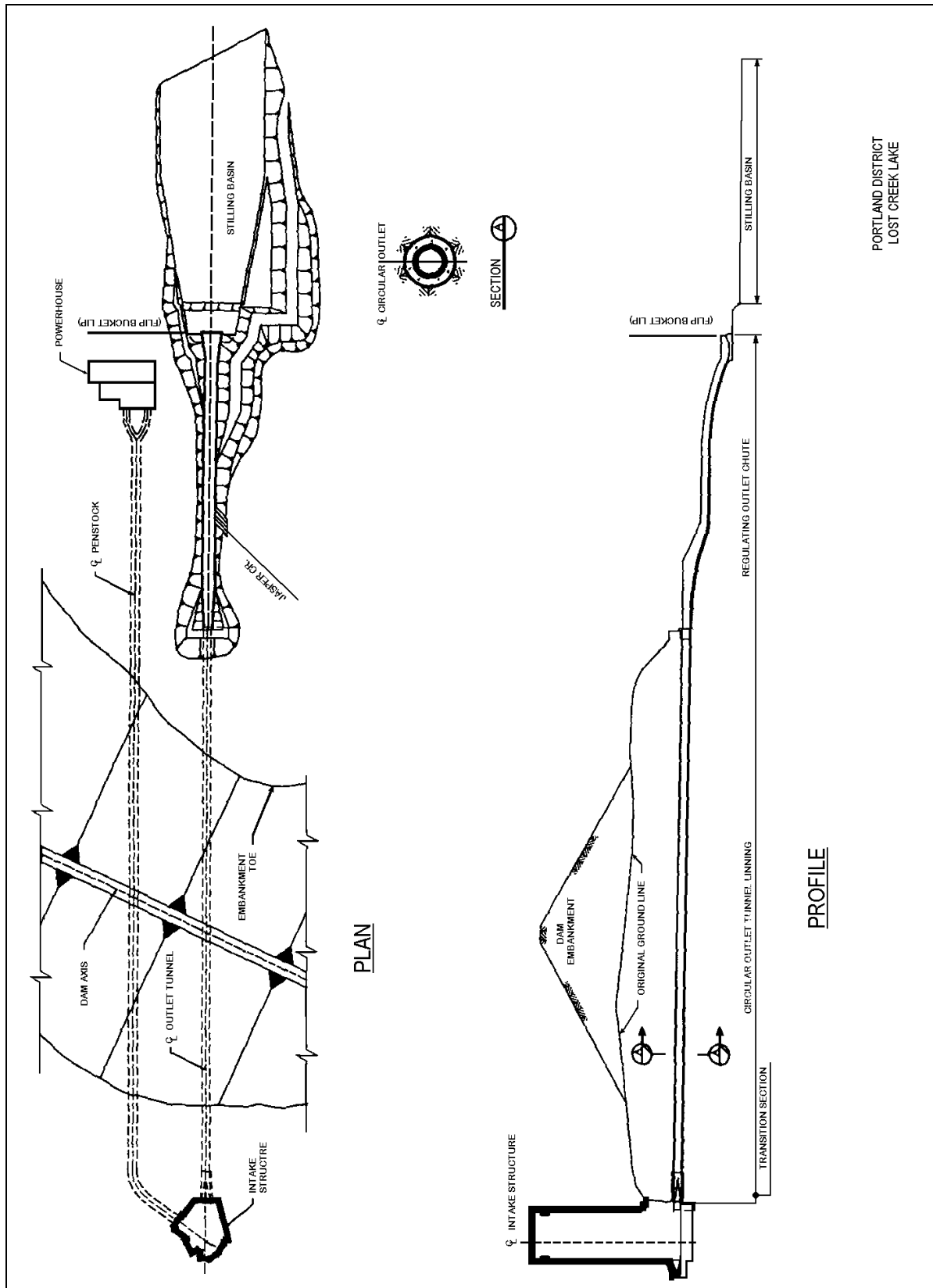


Figure 2-1. Outlet works through abutment, Lost Creek Lake, Oregon

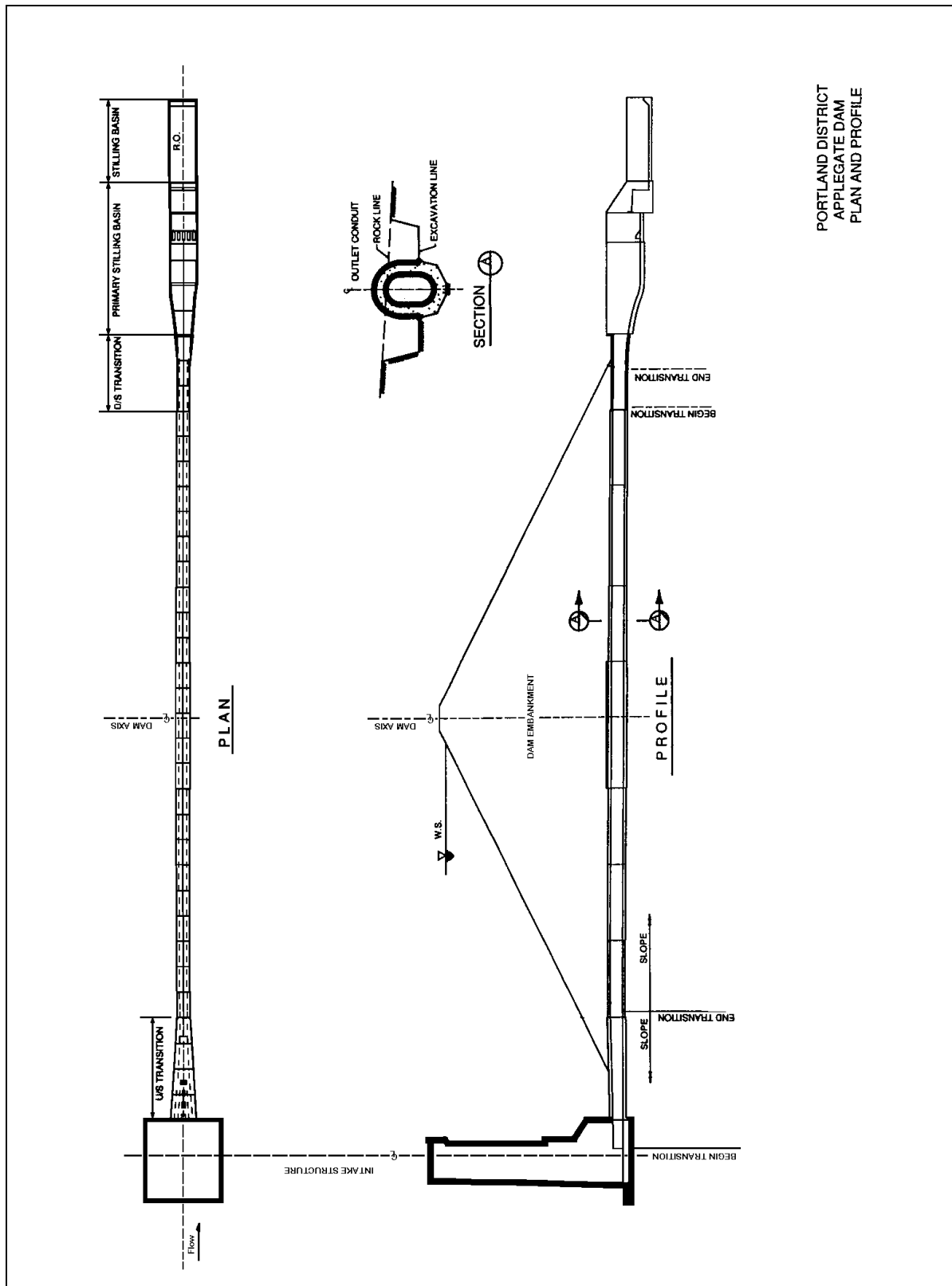


Figure 2-2. Outlet works under embankment dam, Applegate Lake, Oregon

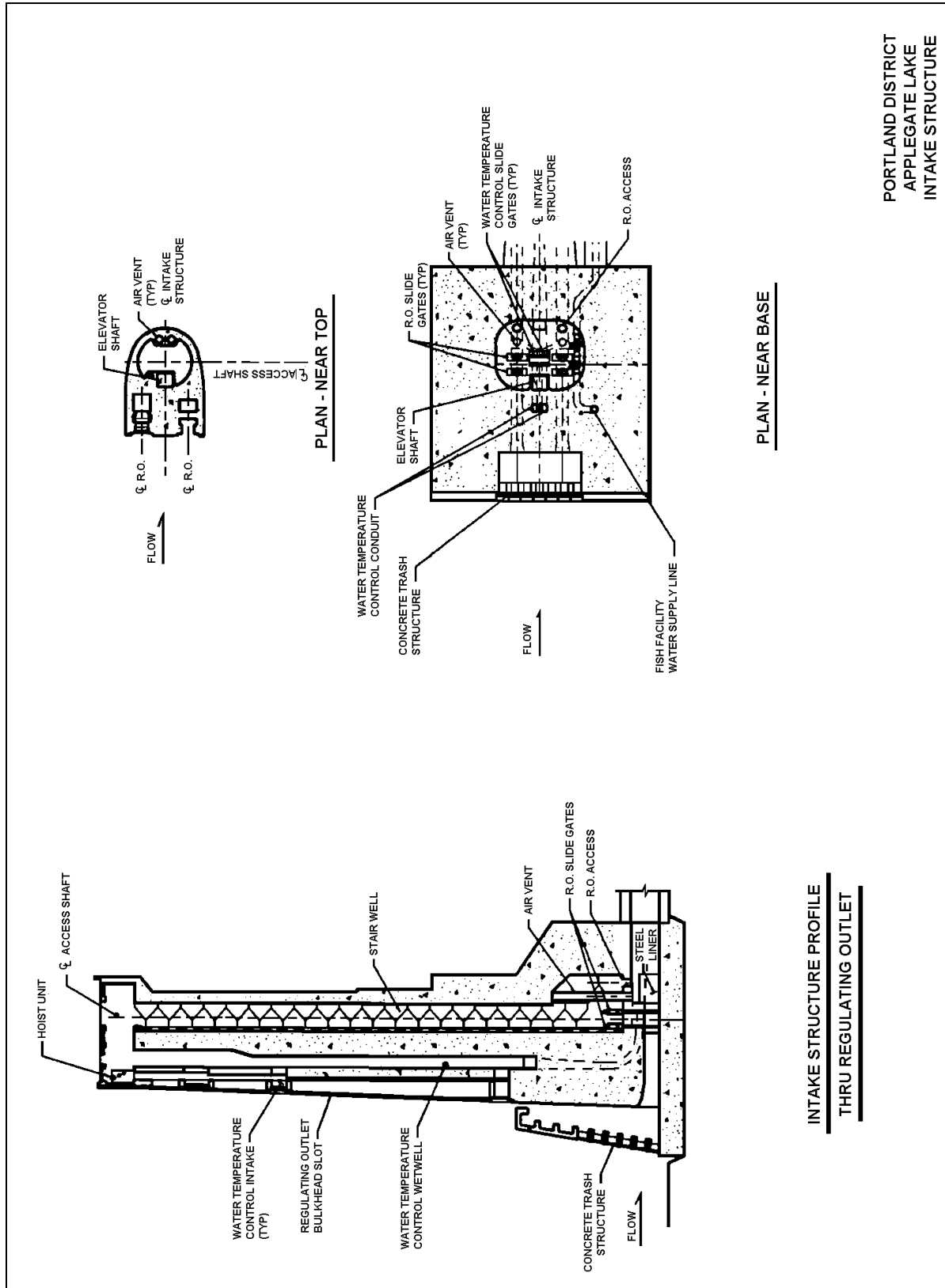


Figure 2-3. Vertical intake structure, Applegate Lake, Oregon

d. Control gating. Reservoir outlet works may be gated or ungated, depending on the project functions. Gated outlet works are needed for multiple-purpose reservoirs that provide storage for conservation, power, irrigation, etc. Gated outlet works are used also for single-purpose flood-control projects in which positive control of discharge regulation is required. Ungated outlets are adapted to some flood-control reservoirs, particularly small ones, where predetermined discharges (varying with the head) will meet the flood-control requirements. In general, the size of gates and conduits should be based upon diversion, evacuation, and operational requirements. A careful study and analysis should be made of the downstream channel capacity. The outlet works capacity should be adequate to discharge downstream channel capacity flows when the reservoir level is at the bottom of the flood-control pool. Consideration should be given to the time required to empty the reservoir for projects requiring emergency drawdown capability. The effect of rapid drawdown on materials at the reservoir margin must be considered. For gated structures, the number and size of gates should be such that one inoperative gate must not seriously jeopardize the flood-control function of the project. The use of two-gate passages is preferred for economy unless studies indicate the operating characteristics would be materially improved by the use of three-gate passages, or unless a wider structure is required for stability or structural reasons. The size of the gates should be based on an individual study for each project evaluating both the release capacity and downstream damage. The height of gates should generally be the same as the diameter of the tunnel or conduit in order to simplify the transition between the gated intake and the downstream tunnel or conduit. It is unlikely that large service gates will be able to regulate small flows with sufficient accuracy. This means, where low flow discharges are required, either a small gate within the service gate or a separate gate for small flows will be required. Usually, a separate gated system for low flow water and water supply regulation is the most reliable. In general, for medium height dams (heads between 80 and 120 ft) the most economical type of gate structure is one located at the upstream end of the outlet works consisting of a wet-well type structure with two gate passages, each passageway regulated by a hydraulically operated caterpillar or wheeled service gate with a single transferrable emergency gate operated by a traveling hoist located in the superstructure at the top of the intake tower. For high-head projects (heads over 120 ft) and for low-head projects (heads less than 80 ft), the most economical type of gate structure will often be different from that used for medium height dams. In any case, alternative studies will be required to verify that the type of outlet works structure selected is the most economical.

e. Exit chute and energy dissipater. Water flowing from the outlet works to the downstream river channel will be at high velocity. Energy-dissipating structures are commonly required to reduce flow velocities to levels that will not cause scouring, which can undermine tailrace structures and damage the downstream channel and riverbed. Energy-dissipating structures for outlet works may be similar to those found on spillways, such as hydraulic jump stilling basins or flip bucket plunge pools. Where releases are small, baffle block structures, riprap scour protection, or cut-off wall scour protection may be all that is required.

f. Approach and discharge channels. The purpose of the approach channel is to convey water into the intake structure. The discharge channel is designed to direct flows back to the river channel. The dimensions and alignment are determined by the hydraulic requirements and the stability of the excavated slopes.

2-2. Intake Structure Types

There are no standard designs for intake structures. Each design is unique and may take on many forms and variations. The intake structures discussed in this manual can be separated into two broad categories: free-standing and inclined. Selection of the appropriate type depends on a number of considerations including site conditions, economics, and effectiveness in meeting project requirements. Project requirements can include reservoir operating range, drawdown frequency, discharge range, trash conditions and required frequency of intake cleaning, reservoir ice conditions, water quality and temperature operating requirements, and environmental requirements such as fish passage. An intake structure may be submerged or may extend above the maximum reservoir water surface, depending on its function. Above-reservoir intake structures are

necessary when gate controls are located on top of the structure, access to an internal gate control room is through the top of the structure, or when operations such as trash raking, stoplog or bulkhead installation, and fish screen cleaning are required from the structure deck. The submerged intake structure is primarily found at low-head, flood-control projects where submergence occurs only during flood periods or at projects where trash cleaning is not required. Submerged intake structures may also consist of simple submerged shafts and horizontal intakes equipped with a trash structure and bulkhead slots.

a. Free-standing structure. The most common type of intake structure is the vertical structure, generally referred to as a free-standing intake tower. It allows increased flexibility when locating the outlet works at the site. The vertical tower is usually more economical and easier to lay out than the inclined intake structure. Conduits and openings, operating equipment, and access features lend themselves more readily to arrangement in the vertical structure. A service bridge provides access to the top of the structure. Figure 2-3 shows a typical free-standing intake tower.

b. Inclined structure. For higher embankment dams in high seismic risk areas where a vertical structure may not be feasible, an inclined intake structure supported against the abutment is an alternative for consideration. An inclined structure has the advantage of increased stability over a vertical structure. In high-risk seismic locations and on steep abutment slopes, anchoring of the structure to the abutment to maintain stability and prevent liftoff should be investigated. Figure 2-4 depicts an inclined intake structure.

2-3. Functional Considerations for Outlet Works Facilities

The type and design of the outlet works facility will be greatly influenced by the project, its purposes, and structure-specific functions. An evaluation of each function is absolutely necessary in any design of the structure selected.

a. Flood control. Outlet works for flood-control projects generally require designs having large flow capacities and less regulation capabilities. Typically, the outlets are gated for flow regulation. However, the conduits may be uncontrolled (no gates) for reservoirs that are low or empty during non-flood periods.

b. Navigation. Projects releasing flows for downstream navigation usually involve lower discharge capacities than flood-control works. Requirements for close regulation of the flow and continuous releases are characteristic with outlets designed for navigation.

c. Irrigation. Gates and valves for irrigation require close regulation and lower discharge ranges than flood-control outlet controls. Releases may be discharged into a channel or conduit rather than into the original riverbed.

d. Water supply. Municipal water supply intakes are generally a secondary project function. Reliability and water quality are of prime importance in the design. Water intakes are located and controlled to ensure that the water is free of silt and algae, to obtain desired temperatures, and to allow intake cleaning.

e. Power. Power penstocks within intake structures should be located so as not to cause any undesirable entrance flow conditions such as eddies that might jeopardize turbine operation. Power intakes may require smaller trashrack openings to limit the size of debris that enters the penstock. Requirements for power intakes are covered in EM 1110-2-3001.

f. Low-flow requirements. Low-flow releases are required at some projects to meet environmental objectives, downstream water rights, water supply, etc. Smaller conduits and pipes separate from the main outlet conduits may be required to maintain minimum releases. Intake trashracks should be sized to prevent plugging of the smaller lines.

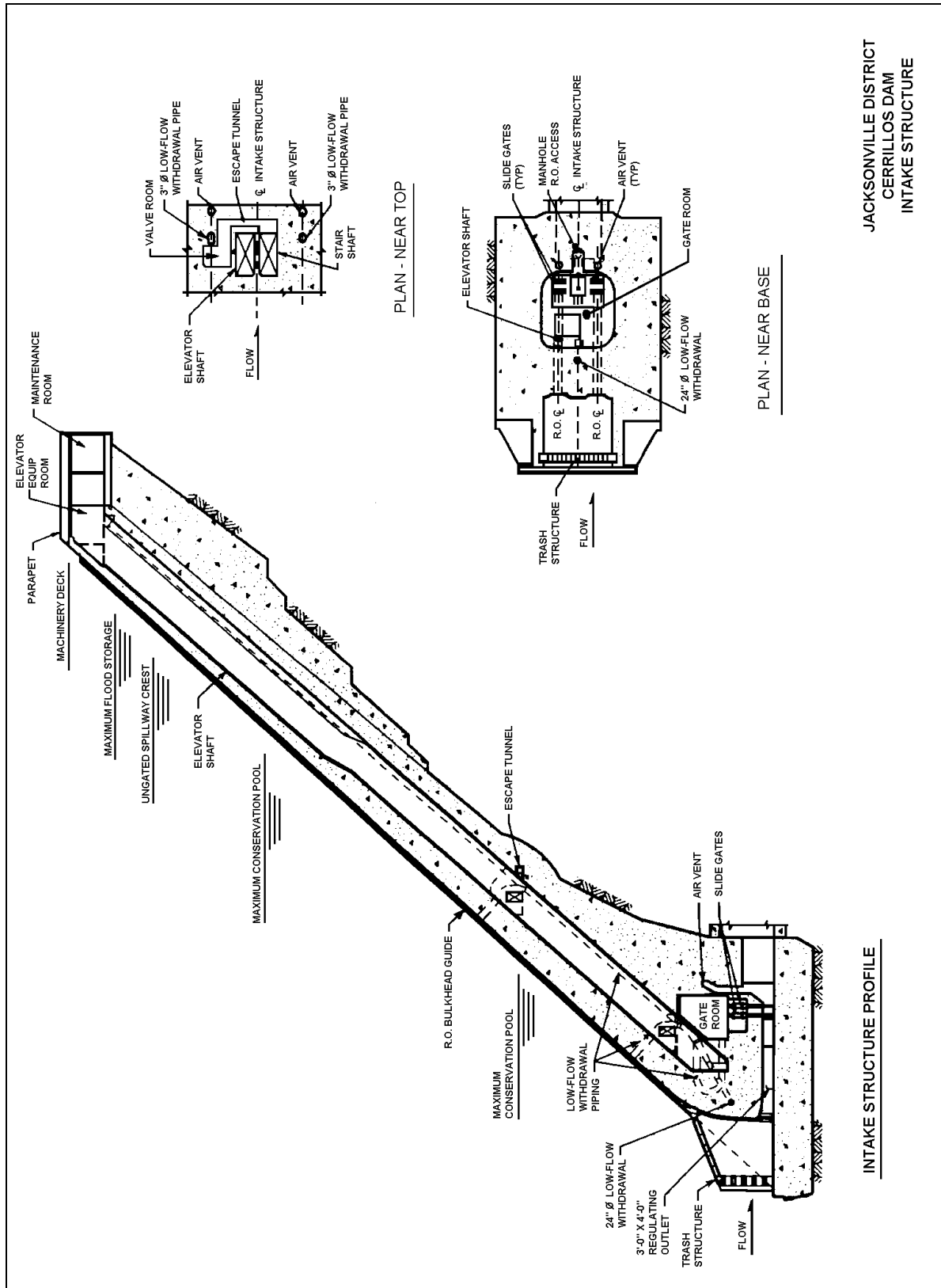


Figure 2-4. Inclined intake structure, Cerrillos Dam

g. Water temperature. Project requirements for fish and wildlife may involve intake features that control water temperature downstream of the reservoir. Multilevel ports and skimmer weirs in conjunction with a mixing wet well may be used to achieve this function. These features are incorporated separately or in combination with the other functions of the intake outlet works.

h. Sediment. Projects designed for sediment retention should be designed to pass flows as the sediment level rises in the reservoir and to prevent sediment from passing through and damaging or blocking both the entrance and the outlet works itself. Smaller releases are controlled by multilevel intakes that are closed by gating or stoplogs as the sediment level rises in the reservoir. For projects with flood flows that are released through the outlet works, a high-level intake that provides protection against large sediment buildup may be necessary.

i. Drawdown. ER 1110-2-50 requires that all new projects have low-level discharge facilities for drawdown of impoundments. Chapter 5 provides further discussion of this requirement.

j. Diversion. Outlet works may be used for total or partial diversion of the river during construction of the dam. The outlet works must be sized for both diversion and final project releases. A cut-and-cover outlet conduit that is used for diversion is typically located outside the normal river channel to facilitate ease of construction. Except for some high-head dams, it usually is economical to place an outlet tunnel near the river channel elevation, allowing its use for diversion. The scheduling and staging of the outlet works in conjunction with the dam construction require careful evaluation. This is particularly important when second-stage outlet works construction needs to be coordinated with dam closure and the ongoing diversion.

k. Outlet diversion with a high-level intake. An alternate plan used for some deep reservoirs includes a high-level intake structure at the head of an inclined tunnel connecting with a tunnel through the abutment near river level. After the river-level tunnel is used for diversion and construction is completed, the low-level tunnel is plugged at the upstream side of its intersection with the inclined tunnel and downstream portion of the low-level tunnel with a connecting curved transition forming the permanent outlet. Figure 2-5 shows an example of this plan. This plan may be economical and satisfactory under special conditions in which the reservoir is very deep and the site provides sound rock for supporting the intake structure and the complex tunnel construction. However, foundation investigations and comparative studies should be performed. The design will require low-level drawdown facilities as outlined in ER 1110-2-50. This plan should not be selected without full analysis of its practicability and economy.

2-4. Intake Tower Design Considerations

a. General. The outlet works is an integral part of a project that includes a dam and spillway. Its layout and configuration therefore should be associated with the planning and development of the complete project. In all cases, selection of the best overall plan for the outlet works should be made after careful comparative studies of alternative plans and consideration of the site conditions. Functional and service requirements, component interrelationships and compatibility, economy, safety, reliability, and repair and maintenance requirements should all be considered in the studies. Site conditions include topography, climate, foundation, geology, and seismicity. Hydrology and minimum flow requirements are important for determining the range of design releases for the outlet and diversion conditions. All operation and maintenance requirements must be identified in order that a safe, reliable, and economical outlet works will be designed. After all the purposes of the project are established and the functions and criteria for the outlet works have been clearly defined, the geometry and layout of the intake tower can proceed. Multiple alternatives should be developed and evaluated to determine the optimum plan.

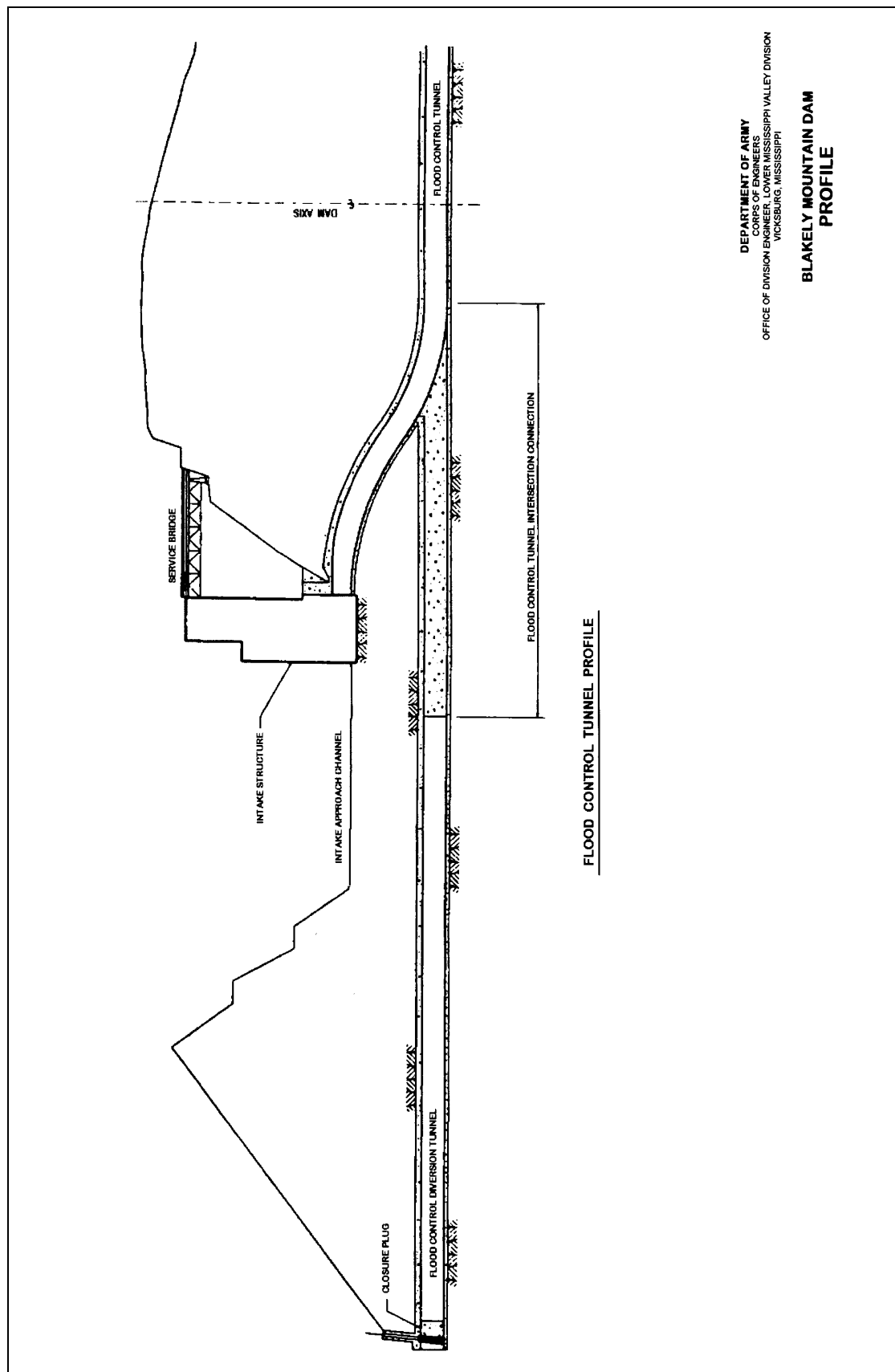


Figure 2-5. High-level intake, low-level diversion tunnel, Blakely Mountain Dam

b. Design considerations and criteria. The intake structure size and configuration are based on the inlets and collection wells size, control gate passages, inlet hydraulic configuration, exit passages, structural element size, and the space and clearance requirements for the mechanical and electrical equipment. The intake structure may take on numerous shapes and forms for each potential alternative. The final configuration will take the effort of a fully coordinated team of structural, mechanical, electrical, material, cost, and geotechnical engineers; a geologist; and hydrological and hydraulic engineers to ensure that all engineering and geological considerations are properly integrated into the overall design. When water quality is a consideration, reservoir hydrodynamics and environmental studies are important in the design process. Mathematical and/or physical hydraulic modeling may be required to support the hydraulic and environmental design.

c. Structural design considerations. After the design layout has been selected, the structural engineer should evaluate the structural integrity. One primary responsibility of the structural engineer is to ensure that the structural design meets the objective of carrying design loads from the top of the structure to the foundation. Wet wells with multilevel ports and weirs must be checked for structural stability, materials, sizing of members, connection details, reinforcement layout, and geometrical compatibility with adjacent features. The objectives of effective structural planning are to maintain symmetry, minimize torsional effects, provide direct vertical paths for lateral forces, and provide a proper foundation. A continuous load path or paths with adequate strength and stiffness that will transfer all forces from the point of application to the final point of resistance must be provided.

d. Siting. Selection of the location for an intake structure depends on multiple factors, including foundation approach conditions, alignment of conduit, and tunnel access. For an embankment dam, the structure is often located adjacent to a hillside into which the outlet works pass. For a cut-and-cover-conduit, the structure is generally located at the upstream toe of the embankment dam. The location and configuration of the structure will also depend on whether the structure will be vertical or inclined. For a vertical intake structure, the location will be determined largely by the design layout of the rest of the outlet works and the dam. However, adequate information about the foundation conditions should be obtained prior to the final site selection to ensure that the foundation will be suitable. For an inclined structure, the selection of the location of the base and the inclination angle are determined primarily by the ground profile and properties of the foundation materials present, e.g., depth of rock weathering, bearing strength, etc.

e. Water intakes. The water intakes are basically positioned with respect to the range of reservoir levels and to meet particular operating functions (project purposes). Low-level reservoir evacuation, sediment deposition levels, and minimum power generation are other criteria for consideration. Intake layout should consider the design of transitions, branches, heads, etc. A trash structure is required for most intakes to protect gates, valves, or turbines. The sizes of the trashrack openings are governed by the minimum waterways opening and the gate size. Further details on the trashracks and their design are provided in Chapter 5.

f. Locations of control gates. The control gates at U.S. Army Corps of Engineers projects are usually located in the intake structure, in a shaft or gate chamber near the extended axis of the dam, in the abutment under or within the dam section, or in one or both of the abutments. Under special conditions, the control gates may be located at the downstream end of a pressure tunnel. The principal advantages and disadvantages of alternative locations for the control gates are discussed in the following paragraphs. Control gates along the outlet and at the downstream end are discussed only briefly. The discussions are applicable to outlet works without diversion and to those with diversion.

(1) Control gates at upstream end. When the gates are placed at the upstream end of the outlet works, the emergency gates, bulkheads, service gates, and trash structures are all combined into a single structure. Closing of the gates allows for inspection and repair of the entire tunnel and for readily removing accumulated trash, sediment, or other deposits. A combined structure is particularly advantageous in cases in which

the permanent conservation or maximum power pool elevation is well above the river level. When other locations are selected for the gate structure, a separate structure for trash removal and bulkhead closures would generally be more expensive. With the control gates at the upstream end, the internal hydrostatic pressure in the tunnel is less than the weight of the overlying rock, earth, and water at all points along the conduit/tunnel profile. A disadvantage with upstream control gates is the cost of extending the structure above the pool and requiring an access bridge. The upstream gate location is preferred when the cost is nearly the same or even slightly higher than that for other gate locations. The upstream closure of the outlet conduits with the control gates provides additional dam safety. The types and limited sizes of gates or valves suitable for use under high heads sometimes require multiple gates in wide intake structures and long transitions between the gates and the outlet tunnel or conduit.

(2) Control gates near dam axis. Placing gates in a gate chamber or shaft in the abutments provides increased protection at high-seismicity sites. Disadvantages include the requirement of a separate bulkhead closure at the upstream end to provide the capability of dewatering the upstream portion of the tunnel for inspection, access for operation and maintenance, and designing for high earth loads where the gate is located in an embankment dam. Tunnels and conduits upstream of the gate chamber will need to be designed for full internal hydrostatic heads.

(3) Control gates at downstream end. Full internal hydrostatic pressure over the entire outlet tunnel requires a steel lining inside the concrete lining to prevent unsafe hydrocharging through cracks in the concrete liner of the surrounding rock near the downstream end. The length of the steel lining will depend on the geologic conditions and the depth of rock over the tunnel. This requirement generally makes this alternative less economical. The downstream location may be favorable in special conditions such as high-head projects and short outlet tunnels. As with the location along the dam axis, upstream bulkhead closure is required.

g. Intake structure shape. There are numerous geometric shapes and combinations that can be developed in the design of an intake structure. The elevational profile can be uniform or tapered, and the plan can be rectangular, circular, or irregular. The following paragraphs discuss three basic geometric shapes.

(1) Rectangular. Rectangular intake structures are more functional for low-head reservoirs that are designed for large discharges. A rectangular shape provides for more efficient layout of entrances and openings, gates and operating equipment, and other features. Rectangular intake structures are usually more easily constructable and site-adaptable. Figure 2-6 shows a rectangular intake structure.

(2) Circular. Circular intake structures are structurally more efficient, providing economic savings, particularly in high-head projects. Hydraulic and access requirements can readily be adapted to the circular shape. The lower section of the structure may be rectangular to provide for arrangement of the water intakes, trashracks, and bulkheads.

(3) Irregular. An irregular design may result in the development of very complex and unusual shapes. Structures with wet and dry wells, high-level intakes, multiple wet wells, fish entrances, and other special features result in numerous different and unusual configurations. Figure 2-7 shows an irregular intake structure.

h. Wet well. Wet wells of intake structures are used to withdraw water from different levels of the reservoir and for mixing to achieve specified controlled temperature releases. Wet wells are also used for multi-port sediment retention where ports are closed with stoplogs as the sediment level rises in the reservoir. Wet wells can be sized as a single unit or several wells.

i. Dry wells. Dry wells, also referred to as access shafts, provide access to the gate room when located near the base of the intake structure. Features commonly incorporated into a dry well are elevator, stairs, air

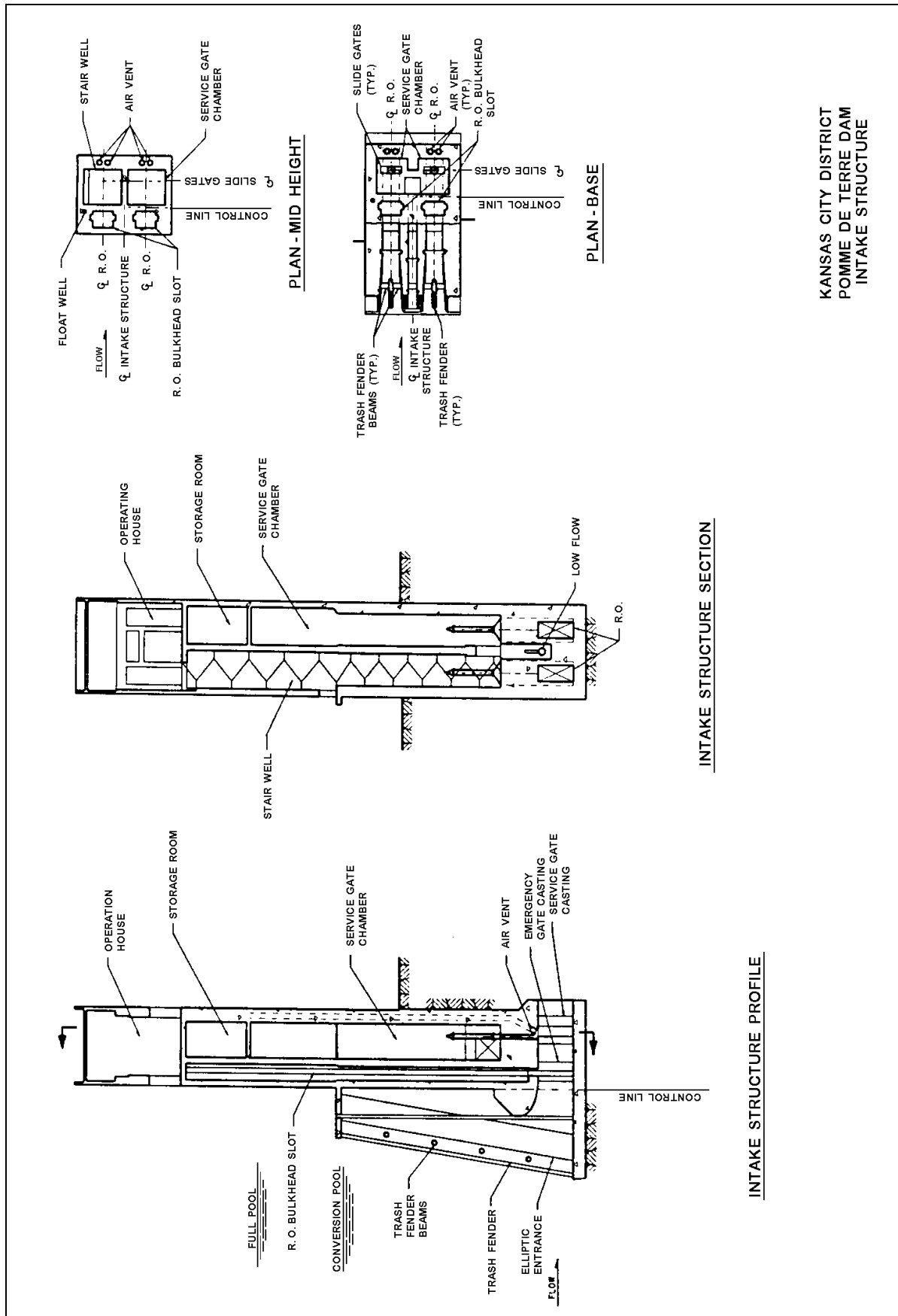


Figure 2-6. Rectangular intake structure, Pomme De Terre Dam

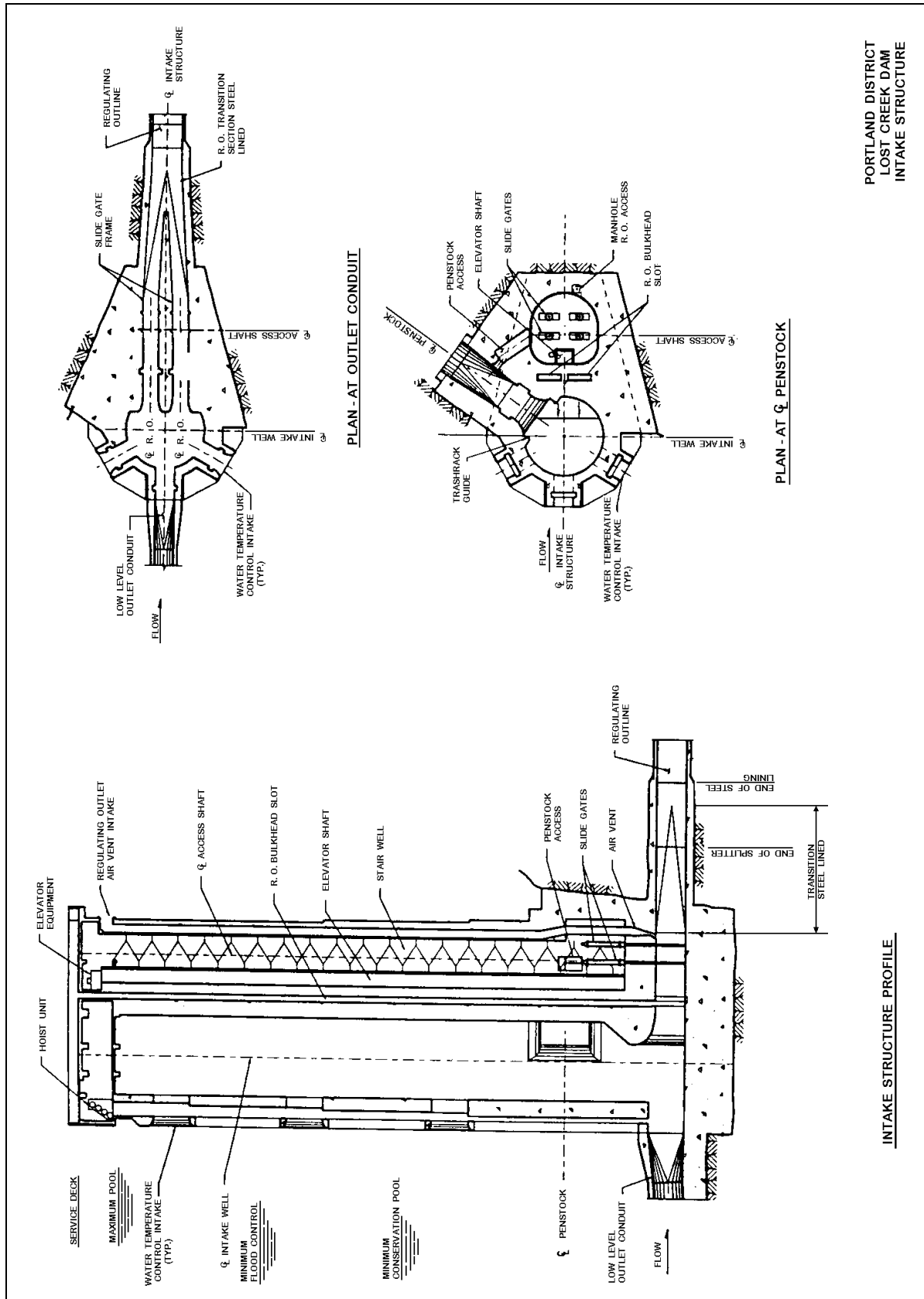


Figure 2-7. Irregular intake structure, Lost Creek Dam

vents, and an open shaft area for the removal and installation of equipment for repair and maintenance. These features should be arranged in the most efficient manner to minimize the dry well size. The open shaft area in the dry well should be sized to allow removal of gates and other gate room equipment.

j. Gate room. Gate room design with both service and emergency gates is outlined in this paragraph. When hydraulically operated slide gates are used, the service and/or emergency gates are usually located in a room in the base of the dry well. This arrangement is applicable to a single-conduit outlet works for a structure at the upstream end of the outlet works, a shaft near the dam axis, or a gate structure near the downstream end of the conduit. A circular-shaped gate room may be advantageous, especially for a high structure. When caterpillar or wheel service and emergency gates with downstream seals are used, the typical gate structure for a single-tunnel outlet works is of the wet well type (usually rectangular in horizontal section) with partition walls between the gate chambers dividing it into separate gate wells. The partition walls make possible the dewatering of any service gate by placing a single emergency gate in a guide upstream of the wall. This type of structure is used also for mechanically operated slide gates with gate stem extensions but is economical only for low-head structures. The water passages at the bottom of the structure are made with gradual transitions at the entrance and downstream from the gates. The intermediate and exterior walls of the passages are made the minimum thickness required to accommodate the gate frames and to withstand the hydraulic forces and structure loads. Minimum access provisions between the operating deck and base of the gate structure should consist of a ladder. In a dry-well structure, a stairway to the gate chamber is desirable. When the gate chamber is located more than 50 ft below the operating deck, an elevator of minimum size and capability is reasonable in addition to the stairway.

k. Basic architectural considerations.

(1) Functional design. Architectural design should focus on meeting operational, safety, and material requirements to include the following:

(a) Access/egress to/from the structure and movement within the facility shall comply with the latest edition of the NFPA 101, Life Safety Code. This includes design of such items as guardrails, walls, partitions, corridors, stairs, doors, etc. Ladders shall be designed in accordance with the requirements of EM 385-1-1, Appendix J. Compartmented storage or other specialized space features shall be incorporated in accordance with the latest edition of the International Code Council (ICC), International Building Code (IBC), and National Fire Protection Association (NFPA) requirements.

(b) System and materials should be selected based on durability, maintainability, economy, and aesthetics. Materials should conform to the Corps guide specification requirements and to nationally recognized industry standards. Selection should consider availability and future replacement to minimize first costs and costs associated with maintenance and repair. Exterior components should be selected and located where damage by vandals will be precluded.

(2) Aesthetic design. Locale must be considered in determining the exterior visual design of an intake tower. Geographical effects such as prevailing weather patterns should be considered in placement and protection of access points or other openings. The relationship of the tower to other existing or proposed structures should be considered and a compatible style or details developed. Remote sites with little opportunity for public access and limited visibility require less architectural treatment than sites where the tower can easily be viewed or visited by the public. When surface treatment of concrete is being considered, EM 1110-1-2009 should be followed.

2-5. Outlet Tunnels Design Considerations

a. General. The location of an outlet tunnel or tunnels is affected by topography, abutment foundation conditions, hydraulic requirements, and economy of construction. Except for some high-head dams, it usually is economical to place an outlet tunnel near stream elevation and use it for diversion during construction. A tunnel should be located far enough into the abutment to obtain adequate cover for the character of rock encountered, and the tunnel alignment must meet the hydraulic requirements. The most economical location to pass around or under the dam and meet the foregoing requirements should be selected. An alternate plan used for some deep reservoirs has a high-level intake at the head of an inclined tunnel connecting with a tunnel through the abutment near river level (see Figure 2-6). After serving its purpose of diversion during construction, the low-level tunnel is plugged at the upstream side of the intersection and the inclined tunnel and downstream portion of the low-level tunnel with a connecting curved transition from the permanent outlet (see Figure 2-5). This plan may be economical and satisfactory under special conditions where the reservoir is very deep, the minimum water surface is high, and rock foundation is adequate. Because of the difficult foundation and structural problems involved in this type of construction, thorough foundation investigations and comparative studies should be made and this plan should not be selected without full analysis of its practicability and economy. The advantage of the high-level intakes for a very deep reservoir is the lower intake structure and lower head on intake gates, resulting in a less expensive intake structure. Disadvantages of this type of outlet are: cost of the inclined tunnel, difficulty of obtaining an adequate foundation for the intake structure, and the difficulty of excavation for and the complexity of the transition structure at the junction of the inclined tunnel with the low-level tunnel. Particular care should be taken to obtain high quality concrete, smooth gradual curves, and good alignment at the junction. If the intake structure is located directly over the low-level tunnel, there should be an ample thickness of good, sound rock between the intake base and the tunnel to transmit the foundation loads to the main rock mass. Sometimes conditions may be favorable for locating the intake far enough to one side of the low-level tunnel to simplify the problem of supporting the intake structure. For some high dams, outlet works separate from diversion facilities and located much higher than the river channel may be suitable. In such cases, overall economy may result from use of separate high-level outlets and use of the downstream portion of the diversion tunnel as part of the spillway outlet.

b. Number and size of tunnels. The proper selection of number and size of outlet tunnels depends on hydraulic requirements, the maximum size of tunnel which it is practicable to drive in the rock encountered, operating flexibility, and economy of construction. In some instances it may be necessary to limit the size of tunnel in yielding rock to prevent excessive movement in the surrounding rock. Generally the larger the tunnel within practicable driving limits, the more economical it is per unit flow capacity. Also, the approach and discharge channels and the intake structure usually are more economical for fewer tunnels because the reduced width results in less channel excavation and a narrower intake structure. However, if it is necessary to use two parallel gates in each larger tunnel and only one in the smaller, the intake structure may not be reduced and economy may not be affected by use of larger tunnels. The most economical number of tunnels which will meet the other requirements should be used. A single tunnel is used when, as is frequently the case, the required diameter to handle the entire discharge is not over 20 ft to 25 ft. In order to provide reasonable operating flexibility, a total of not less than two service gates should be provided in the outlet works. In some cases, this requirement may influence the number of tunnels. Detailed design and construction procedures for tunnels are provided in EM 1110-2-2901, "Tunnels and Shafts in Rock."

c. Approach and discharge tunnels. The dimensions and alignment of the excavated approach channel upstream from the river to the tunnel and of the discharge channel to convey the water back to the river downstream are determined by the hydraulic design requirements and the stable slopes for the material excavated.

2-6. Cut-and-Cover Conduit Design Considerations

a. General. A typical cut-and-cover conduit through an embankment dam is near stream bed level so that it will economically fulfill the hydraulic requirements for the outlet works and serve for diversion during construction. This conduit is placed on suitable rock whenever practicable, as good foundation is of paramount importance to keep settlement as small as possible. The most economical location which provides a good foundation is selected. When practicable the conduit is placed outside the normal river channel to facilitate construction. A location to one side of the river valley often provides the best foundation. Where the conduit sides are placed against rock, presplitting should be used to obtain close control of excavation. Water leaking from or onto the cut-and-cover conduit can lead to piping of embankment dam materials and jeopardize dam safety. Care must be taken when using cut-and-cover construction to assure that the conduit will remain watertight. The use of cut-and-cover construction where the conduit will be pressurized (gate structure near axis of dam or at downstream end) is unacceptable because leakage into the embankment under the higher internal water pressure would be dangerous should a rupture of the conduit occur.

b. Arrangement of multiple conduits. Multiple outlet conduits usually are placed close together so that the intake structure may be the minimum width that will satisfy the hydraulic and structural requirements. They should be planned to minimize cracking by keeping the conduit width to a minimum. This is accomplished by a single multiple-opening conduit structure with longitudinal contraction joints between conduits and generally not more than 50 ft apart. Detailed design and construction procedures for conduits are provided in EM 1110-2-2902, "Conduits, Culverts and Pipes."

c. Approach and discharge channels. The dimensions and alignment of the excavated approach channel upstream from the river to the tunnel and of the discharge channel to convey the water back to the river downstream are determined, in general, by the hydraulic requirements and the location of stable slopes for the material excavated. Concrete retaining walls are provided as needed just upstream from the intake and the trash rack to retain the upstream toe of the embankment on each side of the approach channel and similar retaining walls are provided at the downstream end of the outlet works, if required.

2-7. Energy-Dissipating Structures Design Considerations

Most often the type of energy-dissipating structure needed will be determined by hydraulic model studies. These studies should also be used to determine the loadings to be expected on the energy-dissipating structure during all flow conditions up to the Maximum Design Flood (MDF). Energy-dissipating structures may consist of (a) abrupt expansions in high pressure conduits, (b) hydraulic jump-type stilling basin structures, (c) flip bucket plunge pool structures, valves, and deflectors which spray jets into the air before entering a plunge pool structure, and (d) various baffle block energy-dissipating structures. The stability of the various energy-dissipating structures will be determined in accordance with EM 1110-2-2502. Information on spillway-type stilling basins can be found in EM 1110-2-2200. The structural design of energy-dissipating structures will be in accordance with the provisions of EM 1110-2-2104. The hydraulic design requirements of these structures can be found in EM 1110-2-1602.

2-8. Coordination Among Disciplines

The structural engineer should be involved in technical coordination of structural features during all project phases. Structural design activities should be coordinated with other functional elements of geotechnical, hydrology, hydraulic, mechanical, electrical, and architectural design, construction, operations, cost engineering, real estate, surveying, mapping, etc., to develop the design of the structural features. Technical coordination should be maintained with the technical staff of the local sponsor. Also, technical coordination with higher authority is encouraged to reach early agreement on unprecedented or complex problems.

Chapter 3 Outlet Works Design

3-1. General

a. Summary. This chapter presents discussion and guidance on the concrete and foundation properties generally considered in design, the loads and load conditions typically applied to outlet works intake towers, and various other information on stability analysis and structural design. Design and load conditions of the intake structure operating components and appurtenant features are discussed in Chapters 5 and 6.

b. Structural design. The structural design should be developed in conjunction with the hydraulic design to provide a stable, load-resistant structure which meets all functional and structural performance requirements. Structural elements of the intake structure should be sized and proportioned to resist the various load conditions that will be imposed during construction and operation. The structural design and geotechnical conditions at the site must be closely coordinated.

c. Stability. Initial layouts are followed by a stability analysis. If the structure fails to meet criteria, the layout is modified and reanalyzed. This process is repeated until an acceptable configuration which meets stability criteria for all load conditions is attained.

d. Strength, serviceability, and dam safety. Outlet works structures must be designed for strength and serviceability and to protect the project against potential latent deficiencies that could jeopardize dam safety.

(1) The structure strength must be sufficient to provide the required margin of safety for all possible loads and load conditions. Serviceability means the structure is watertight and resistant to potential damage from cavitation or abrasion erosion and is adequately protected against corrosion of reinforcing steel, embedded steel, and exposed steel components.

(2) The possibility of upstream ground slope instability due to floods or earthquakes should be taken into account during the design of the outlet works. During a major flood or earthquake, the ground slopes upstream of the project may become unstable causing landslides and the accumulation of large quantities of debris and sediment at the outlet works entrance. The configuration of the approach and approach walls should minimize the potential for sediment and debris accumulation which could block the outlet works entrance. Provisions should be made to allow for removal of debris and sediment that could accumulate at the outlet works entrance.

(3) Major floods and earthquakes can be catastrophic to lifelines. Roads, bridges, electrical power, and communications are necessary to respond quickly to an on-project emergency condition. The protection of project lifelines important to operation of the intake/outlet works should be provided for during design.

(4) Leakage from the outlet conduit or along the conduit/dam interface for cut-and-cover conduits can lead to slope instability at critical abutment areas and at the embankment dam. The intake structure/conduit layout and design should protect against any such deficiencies that might jeopardize dam safety.

3-2. Design Data

a. Concrete properties. Concrete properties normally used to design intake structures include the unit weight; compressive, tensile, and shear strengths (friction and cohesion); Young's modulus of elasticity; and Poisson's ratio. Mass concrete structures that require thermal studies to assess the effects of stresses induced by temperature changes in the concrete require thermal properties which include conductivity, thermal

diffusivity, specific heat, and the coefficient of thermal expansion. In structures that are designed to pass sediment, trash, or other debris, special high-strength concretes such as silica fume concrete may be utilized to provide highly abrasion-resistant surfaces.

b. Foundation properties. The principal foundation properties for outlet works design are bearing capacity, deformation and elastic modulus, and shear strength. These properties determine suitability of the rock mass to support the structure placed on it and the resistance to sliding at the structure/foundation interface. Dynamic elastic modulus and shear strength are important factors in the seismic design of these structures. Also very important to stability analysis is whether any weak zones exist within the foundation where sliding failure could occur. Such zones may consist of shears, faults, or other discontinuities along which material of low strength such as clay gouge may be present in an otherwise sound rock mass. Significant voids such as those produced by dissolving of some limestone formations also influence foundation suitability. The foundation properties must be investigated early in the design phase of the project to determine design parameters and any remedial treatment that may be necessary during excavation and construction. The structural design and contract drawings should include provisions for unforeseen conditions which may require that the intake structure be founded at a lower elevation in order to reach competent rock.

3-3. Loads

The following forces are typical of those that may be required in the stability and stress analysis:

a. Dead load. The dead loads considered should include the weight of the structure and appurtenances such as gates, trashracks, bridges, and other attached features.

b. Hydrostatic. The hydrostatic loads against the structure should include internal and external pressures for all design operating conditions as well as for any construction conditions relevant to intake structures designed for diversion.

c. Uplift. Uplift pressure due to headwater exists along the structure-foundation interface, within the foundation below the base, and through any cross section within the structure. Uplift pressure is present within the cracks, pores, joints, and seams in the concrete and foundation material. Uplift at the base should be assumed to be 100 percent of the reservoir pressure over 100 percent of the base area. At internal planes, uplift shall be assumed to vary linearly from the hydrostatic head at the external surface of the intake structure to the hydrostatic head at any internal surface. Uplift pressures should be applied over 100 percent of the area of the plane and are assumed to remain unchanged during an earthquake.

d. Earth and silt. For structures within the dam embankment, earth pressures should be included in the design. Silt pressures are included where sediment buildup around the intake structure can be expected. The silt and earth material may or may not be submerged. The design should include evaluation of the site for liquefaction. The magnitudes of earth and silt pressures are discussed in EM 1110-2-2502.

e. Temperature. Temperature rises and cracking resulting from cement hydration are a concern only in intake structures with large massive concrete sections. Temperature rises caused by cement hydration can cause significant cracking. Cracking can be minimized and controlled through temperature control of mix, lift heights, insulation, and use of reinforcement steel. Temperature control measures are discussed in greater detail in Chapter 6, paragraph 6-4.

f. Earthquake forces. Chapter 4 presents criteria and guidance for determining earthquake forces.

g. *Wind pressure.* Wind loads shall be applied to the structure in accordance with American Society of Civil Engineers (ASCE) 7 (ASCE 1995). Wind loads will also be applied to the appurtenant structures as applicable.

h. *Ice pressures.* In locations where ice is a design consideration, the ice pressure should be applied in combination with operation and maintenance conditions. EM 1110-2-1612 furnishes guidance for magnitude and application of ice loads.

i. *Debris and trash.* Loadings on the structure resulting from debris and trash will depend on the nature of the trash burden in the reservoir, maximum hydraulic differential created by debris buildup on trashracks, the structure layout and operation, and other factors. Chapter 5 further discusses debris loading.

j. *Wave pressure.* Wave pressures will be determined from the fetch of the reservoir as related to the intake structure and the wind velocity and duration. The *Shore Protection Manual* (1984) provides information relating to wave forces.

k. *Operation and maintenance loads.* Cranes, trucks, stoplog and bulkhead installation and storage, and other maintenance equipment are loadings for consideration.

3-4. Load Conditions

a. *General.* The loading conditions provided in Table 3-1 generally apply for checking the stability and structural design of an intake structure. The loading cases for a specific intake structure should be chosen as applicable from the lists below. Specific operational and site conditions from construction through project life and structure configuration may require that the following conditions be modified or that additional analysis of conditions be made.

b. *Basic loading conditions descriptions.* Specific operational and site conditions from construction through project life and structure configuration may require that the stability loading conditions be modified, or that additional analysis of conditions be made. The loading conditions of Table 3-1 are described below.

(1) Loading Condition U1 - normal pool, all gates open.

- Dead load of structure.
- Reservoir at normal operating pool, annual mean maximum pool elevation with a 2-year return period.
- Earth load (if any).
- Ice loads, if applicable.
- Uplift.
- Water surface inside structure drawn down to hydraulic gradient with all gates fully open.
- Wave loads, if applicable.

Table 3-1

Intake Tower Loading Condition Classification

Load Case Number	Loading Description	Loading Condition Classification
U1	Normal Pool, All Gates Open	Usual
U1	Normal Pool, All Gates Open	Usual
U2	Normal Pool, One or More Gates Closed	Usual
U3	Normal Pool, All Gates Closed	Usual
U4	Normal Pool with Silt	Usual
U5	Minimum Pool	Usual
UN1	Infrequent Flood, All Gates Open	U/UN/E
UN2	Infrequent Flood, One or More Gates Closed	U/UN/E
UN3	Infrequent Flood, All Gates Closed	U/UN/E
UN4	Construction	Unusual
UN5	Diversion	Unusual
UN7	Maintenance Bulkheads in Place	Unusual
UN8	Operating Basis Earthquake	Unusual
E1	Maximum Design Earthquake	Extreme
E2	Maximum Design Flood	Extreme

Note: U = Usual; UN = Unusual; E = Extreme.

(2) Loading Condition U2 - normal pool, one or more gates closed.

- Dead load of structure.
- Reservoir at normal operating pool.
- One or more gates closed with other gates fully open and water surface drawn down to hydraulic gradient in remainder of structure in combinations that produce the most unstable conditions.
- Earth load (if any).
- Ice loads, if applicable.
- Uplift.
- Wet well full of water upstream from closed gate.
- Wave loads, if applicable.

(3) Loading Condition U3 - normal pool, all gates closed.

- Dead load of structure.
- Reservoir at normal operating pool.
- Earth load (if any).
- Uplift.
- Wave loads, if applicable.

(4) Loading Condition U4 - loading conditions U1 through U3 with silt (if any).

(5) Loading Condition U5 - minimum pool.

- Reservoir empty or at minimum pool.
- Dead load of structure.
- Earth load (if any).
- Ice loads, if applicable.
- Wind load in the direction that would produce the most severe foundation pressures.
- Uplift.
- Wave loads, if applicable.

(6) Loading Conditions UN1 through UN3. Loading conditions UN1 through UN3 are the same as U1 through U3 except the reservoir level is at the infrequent flood stage (i.e., water is at the top of the spillway gates for gated spillways or at the spillway crest for flood-control projects with ungated spillways).

(7) Loading Condition UN4 - construction.

- Reservoir empty.
- Dead load of structure (partially or fully completed).
- Earth load (if any).
- Heavy construction equipment required on or near the structure during construction.
- Wind load in the direction that would produce the most severe foundation pressures.

(8) Loading Condition UN5 - diversion.

- Reservoir at maximum elevation expected during diversion.
- Dead load of structure at diversion level completion.
- Earth load (if any).
- Heavy construction equipment required on or near the structure.
- Wind load in the direction that would produce the most severe foundation pressures.

(9) Loading Condition UN7 - maintenance bulkheads in place.

- Bulkheads in place, no water in structure downstream of bulkheads.
- Dead load of structure.

- Reservoir at maximum pool level at which bulkheads are used.
- Earth loads (if any).
- Uplift.

(10) Loading Condition UN8 - operating basis earthquake (OBE).

- OBE for the most critical of the conditions U1 through U5 with the reservoir at the elevation that is likely to exist coincident with the selected earthquake event.
- No ice.

(11) Loading Condition E1 - maximum design earthquake (MDE).

- MDE for the most critical of the conditions U1 through U5 with the reservoir at the elevation that is likely to exist coincident with the selected earthquake event.
- No ice.

(12) Loading Condition E2 - maximum design flood (MDF).

- Pool at probable maximum flood (PMF) elevation.
- All gates opened or closed, depending on project operating criteria.

3-5. Stability Analysis

a. General requirements. The basic requirements for the stability of an intake structure and other outlet works structures are described in EM 1110-2-2502. In summary:

- (1) Outlet works structures should be safe against rotational instability at the base, at any plane below the base, and at any horizontal plane within the structure.
- (2) Outlet works structures should be safe against sliding on any plane within the structure, at the base, or on any rock seam within the foundation.
- (3) Outlet works structures should be safe against foundation bearing failures.

3-6. Structural Design

All parts of the intake structure should be designed to meet the strength and serviceability requirements outlined in EM 1110-2-2104. Load factors should be in accordance with EM 1110-2-2104 unless otherwise indicated (Refer to Appendix B for seismic loading condition strength requirements). The reinforced concrete elements of the structure should be designed to withstand the loading conditions given in paragraph 3-4. Additional loading conditions may be required to account for site-specific conditions not covered by paragraph 3-4.

Chapter 4 Outlet Works Seismic Criteria

4-1. General

a. Design and evaluation objectives. Outlet works located in seismic zones 1 through 4 as defined in ER 1110-2-1806 should be seismically evaluated or designed with regard to strength, stability, and serviceability. New intake structures located in seismic zone 0 need not be seismically analyzed unless local site conditions indicate possible response to a nearby seismic event. The criteria set forth in this chapter apply to the structural analysis for earthquake loading of outlet works in accordance with ER 1110-2-1806. The approach accounts for the dynamic characteristics of the tower-water-foundation system, and includes the interaction of the system with the ground motion, gravity forces, and appurtenant structures. The seismic analyses should proceed progressively to produce cost-effective, efficient, and reliable structural configurations and seismic details.

b. Coordination of seismic analysis and design. The design team should normally consist of a structural engineer, a materials engineer, a geotechnical engineer, and an engineering geologist. Such a team should ensure that the seismic analyses, designs, or evaluations are appropriate for the project, and include all aspects of material properties, foundation properties, analysis procedures, and design criteria. The design should be based on historical data, appropriate testing, and engineering judgment. It is essential that the structural engineer be involved in all phases of the material and foundation property assessments, the site seismicity assessment, and the assignment of design ground motions. The basic concepts involved in the seismicity studies must be understood by the structural engineer because he or she must use the results of such studies and therefore must ensure that the information provided is meaningful for structural design or retrofit. In an assessment of the proper level of input, establishing either the level of confidence or the upper and lower bounds will define the sensitive parameters and bracket the results. Such steps should help to ensure that a logical and reasonably conservative design/evaluation of the intake tower could be performed. Assessments should not be based on unnecessary compounding of conservatism because this will lead to an uneconomical design or unnecessary retrofit.

c. Structural stability analysis.

(1) Sliding stability. Preliminary seismic sliding stability can be estimated by the seismic coefficient method. A seismic coefficient equal to two-thirds the peak ground acceleration (PGA) should be used. Shear strength parameters required to perform the sliding analysis should be provided by the engineering geologist or geotechnical engineer. If appropriate testing has not been conducted, an estimated range of values should be provided. The following are minimum required safety factors for seismic sliding analysis:

Operating Basis Earthquake (OBE) = 1.7 for critical structures, and 1.3 for other structures
Maximum Design Earthquake (MDE) = 1.3 for critical structures, and 1.1 for other structures

The computer program CSLIDE may be used to perform the sliding analysis. When rigid body sliding analysis indicates a factor of safety is less than required, a better estimate of safety factor can be made using dynamic analysis or a permanent displacement analysis, if necessary. In the permanent displacement approach the structure is permitted to slide along its base and the accumulated displacement during the ground shaking should be limited to specified allowable values. The analysis method is described in paragraph 2-9a(2) of EM 1110-2-6050. The computer program CSLIP may be used.

(2) Rotational stability. For OBE loading conditions, the location of all forces acting on the base of the structure must be such that 75 percent of the base of the structure is in compression. For MDE loading conditions, the resultant location must be within the base. If the resultant location is outside the base, a

dynamic rotational stability analysis should be performed. Rocking occurs when the foundation does not liquefy and

$$S_A > g(b/h) \quad (4-1)$$

where

S_A = spectral acceleration of the first model

g = gravitational acceleration

b = one half of the base width

h = vertical distance from the base to the center of gravity

If rocking occurs, the tower may not be rotationally unstable during the OBE or MDE. Scaling effects, as discussed in Appendix E, show that if S_A is just sufficient to rotate the block through a critical angle α_{cr} , the block will not overturn unless the spectral displacement S_d exceeds one half of the base width. So if a tall slender block is given an initial velocity S_v , which is just sufficient to rock the block to the point of impending static instability, and for small angles (less than 20 degrees).

$$\alpha_{cr} = \frac{S_v}{\sqrt{gr}} \quad (4-2a)$$

and

$$S_d < b \quad (4-2b)$$

where

α_{cr} = the critical angle of rotation from the initial upright position of static equilibrium to the final tipping position of impending rotational instability, radians

S_v = the response spectral velocity for the OBE or MDE ground motion, cm per second

g = gravitational acceleration

r = radial distance from the edge of the base to the center of gravity of the tower, cm

S_d = the response spectral displacement for the OBE or MDE ground motion, cm

The example of a rotational stability of an intake tower is given in Appendix E.

(3) Bearing pressure. Bearing capacity is analyzed according to paragraph 2-6 of EM 1110-2-2200.

4-2. Seismic Evaluation and Design of Tunnels and Cut-and-Cover Conduits

Seismic evaluations of underground structures, where complex structure-foundation interaction is involved, are often based on the performance of similar structures during major earthquakes rather than on the results of numerical analysis. Underground structures are generally less sensitive to seismic effects than surface structures. Information about the performance of underground structures is relatively scarce, largely

because of the rarity of failures of underground structures during earthquakes. Underground structures, unless very rigid, deform to accommodate earthquake ground displacements due to traveling seismic waves, ground settlement, lateral spreading, liquefaction-induced deformations, and fault movements. Inertial forces due to ground shaking, as well as any deformational effects due to traveling seismic waves, generally are not significant for underground structures. The ground displacement effects due to lateral spreading of an embankment can be significant, but the effect on cut-and-cover conduits can be difficult to predict. Conduits for outlet works must be subjected to significant ground distortions or permanent displacements due to lateral spreading of the dam embankment or crossing an active fault before they would be expected to experience damage that would lead to failure. A concrete shell will be subjected to compression and extension at points on the exterior and interior of the lining. The exterior extension is of no consequence. In the event that tension cracks appear on the interior surface, they will close again after a fraction of a second. Such cracks do not usually extend through the thickness of the concrete and cannot, in themselves, form a failure mechanism. Information on seismic effects on tunnels can be found in EM 1110-2-2901 along with a simplified method for analyzing tunnels in rock for seismic effects.

4-3. Seismic Evaluation or Design of Intake Structure Bridge

Bridges to intake towers will interact with the tower during an earthquake. The type of interaction will depend on bearing restraint conditions. Most restraint conditions cause a nonlinear response between the bridge and the tower. This nonlinear response is difficult to incorporate into a model that includes both the tower and bridge. Guidance used for modeling the response between bridge superstructures and their supports is provided in Federal Highway Administration (1995). This information is also applicable to modeling bridge-tower interaction. An alternate approach is to assign the bridge mass to the tower, piers, and abutment based on a tributary mass approach. In most cases this approach provides reasonable results since the stiffness of the bridge system usually has little influence on tower behavior. Additional guidance on the seismic analysis methods for bridges and their supports is provided in American Association of State Highway and Transportation Officials (1989).

4-4. Seismic Evaluation or Design of Mechanical and Electrical Equipment

Mechanical and electrical equipment should be seismically protected in accordance with the provision of Tri-Service manual TI 809-04 "Seismic Design for Buildings."

4.5. Seismic Evaluation or Design of Basin Walls (BW)

a. Seismic performance. Case studies of damage to or failure of basin walls induced by ground motions during earthquakes clearly show the need for including appropriate provisions in the design and detailing of walls located in zones of moderate and high seismic activity. Damage to channel walls in the 1971 San Fernando earthquake is described in Clough and Fraguszy (1977). Damage was typically associated with backfill settlement and inward tilting of the walls toward the channel with the center of rotation at the connection between the wall and the channel slab. A few monoliths collapsed into the channel, but most damaged monoliths moved only enough to yield the reinforcement and severely crack the concrete. Severe damage to the walls or a basin induced by lateral backfill pressures may impair capability to release water from the reservoir, and therefore basin walls should be considered a vital feature in the overall seismic design process for outlet works and embankment dams.

b. Progressive analysis. Design of basins depends on the site characteristics, foundation conditions, basin width, and discharge capacity of the outlet works. Basin walls may be a gravity or cantilever structure that may be laterally restrained by the basin slab, or a U-frame structure supported on sound rock, highly overconsolidated soil, or piles. Considering the number of potential design parameters and the complex dynamic nature of the backfill-wall-foundation interaction during earthquakes, it is clear that the seismic design or evaluation of basin walls necessitates many simplifying assumptions. The investigation usually

begins with a stability analysis using seismic coefficients and a simplified wedge model. When a rigid body sliding analysis indicates the safety factor is less than required, a permanent displacement analysis should be performed as described in paragraph 4-1d(1). In special cases, a dynamic soil-structure interaction (SSI) analysis of the basin walls may be performed using a finite element model. Deciding to proceed to the next level of investigation depends on if the seismic condition is controlling the design or evaluation, and if the relative cost of obtaining information from a more realistic analysis is a better investment than building reserve capacity into the wall system.

c. Basin walls with backfill. The time-history response of walls with backfill is complicated because in addition to the interaction with the foundation rock and water, the wall is also subjected to the dynamic soil pressures induced by ground shaking. Dynamic backfill pressures are related to the relative movement between the soil and the basin wall and the stiffness of the backfill. Behavior of the basin walls may be controlled by a rocking and/or translational response to earthquake shaking. The type of response will correlate to different distributions of backfill pressure acting against the wall. The appropriate method for analyzing the backfill pressure may be categorized according to the expected movement of the backfill and wall during seismic events.

(1) Backfill yields. The relative motion of the wall and backfill material may be sufficiently large to induce a limit or failure state in the soil. This condition may be modeled by the Mononobe-Okabe method (Mononobe and Matuo 1929; Okabe 1924), in which a wedge of soil bounded by the wall and an assumed failure plane are considered to move as a rigid body with the same ground acceleration. The dynamic soil pressures using this approach are described in Chapter 3, Section IV, paragraph 3-26*b* of EM 1110-2-2502.

(2) Backfill does not yield. For sufficiently low intensity ground motions the backfill material may respond within the range of linear elastic deformations. Under this condition the shear strength of the soil is mobilized at a low level so there are no nonlinear deformations in the backfill. The dynamic soil pressures and associated forces in the backfill may be analyzed as an elastic response using Wood's method as described in Ebeling and Morrison (1992).

(3) Backfill partially yields. The intermediate condition in which the backfill soil undergoes limited nonlinear deformations corresponds to the shear strength of the soil being partially mobilized. The dynamic backfill pressures may be idealized as a semi-infinite uniform soil layer using a constant-parameter, single-degree-of-freedom (SDOF) model (Veletsos and Younan 1994) or a frequency-independent, lumped-parameter, multiple-degree-of-freedom (MDOF) system (Wolf 1995). The dynamic pressures for an irregular backfill may be analyzed using a SSI model such as FLUSH (Lysmer et al. 1975). The wall is usually modeled with two-dimensional (2-D) elements. The foundation rock is represented by 2-D plane-strain elements with an appropriate modulus, Poisson's ratio, and unit weight. Transmitting boundaries in the form of dashpots are introduced at the sides of the foundation rock to account for the material nonlinear behavior with depth. The shear modulus and soil damping vary with the level of shearing strain, and this nonlinear behavior is usually approximated by an equivalent linear method. The boundary conditions for the backfill may also be represented by dashpots. Hydrodynamic pressures exerted on the wall are computed using the Westergaard formula.

d. Simplified wedge method. A seismic coefficient method may be used to estimate the backfill and wall inertial forces as described in Chapter 3, Section IV, paragraph 3-26*c* of EM 1110-2-2502. Theoretically, a wall may behave as a rigid body that is fully constrained along its base and sides by the ground, so all parts of the wall may be uniformly affected by accelerations, which are identical to the time-history of the ground motions. Therefore it would be appropriate to use a seismic coefficient equal to the peak ground acceleration for stability analysis of short, stiff walls. However, field and test data show that most walls do not behave as a rigid body, but respond as a deformable body subjected to effective ground motions. Thus the magnitude of the accelerations in a deformable wall may be different from those at the ground surface, depending on the natural period and damping characteristics of the wall and the shaking

characteristics of the ground motions. Furthermore the maximum acceleration will affect the wall only for a short interval of time, and the inertia forces will not be equivalent to those of an equal static force that would act for an unlimited time, so the deformations resulting from the maximum acceleration will be smaller. Design or evaluation of basin walls for zero relative displacement under peak ground accelerations is unrealistic, so the seismic stability analysis should be based on a seismic coefficient that recognizes that an acceptably small amount of lateral displacement will likely occur during a major earthquake. Experience has shown that a seismic coefficient equal to two-thirds of the peak ground acceleration is a reasonable estimate for basin walls. For partially yielding backfill, the strength mobilization factor should be equal to the reciprocal of the minimum required sliding safety factor for that load case.

4-6. Seismic Evaluation or Design of Intake Towers

a. Critical classifications for intake towers. According to ER 1110-2-1806, critical features of civil works projects are the engineering structures, natural site conditions, or operating equipment and utilities at high hazard projects whose failure during or immediately following an earthquake could result in loss of life. A critical intake tower is as described based on its capability to lower the reservoir. Damage to or failure of an intake tower located at a high-hazard project may result in a reduced ability to lower the pool following an earthquake. Lowering of the pool may be necessary to relieve pressure head on an embankment dam possibly damaged by earthquake ground motions, or to inspect and repair an embankment dam. In cases where the loss of capacity to lower the pool will result in downstream fatalities, the tower is a critical project feature. If these conditions do not jeopardize lives, the tower is not critical.

b. Design earthquakes and performance requirements. There are two design earthquakes used in accordance with ER 1110-2-1806:

(1) Operational basis earthquake (OBE). The OBE is the level of ground motion for which the structure is expected to remain functional with little or no damage. The OBE is defined as a ground motion having a 50 percent probability of exceedance during the service life of 100 years (a 144-year return period). The associated performance level is the requirement that the structure will function within the elastic range with little or no damage and without interruption of function. In a site-specific study the OBE is determined by a probabilistic seismic hazard analysis (PSHA). The PSHA is described in EM 1110-2-6050.

(2) Maximum design earthquake (MDE). The MDE is the maximum level of ground motion for which the structure is designed or evaluated. The tower may be damaged but retains its integrity. The earthquake performance evaluation of an intake tower for the MDE is based on linear elastic analyses. The evaluation may require postelastic analyses with considerable judgment and interpretation of the results and should be done in consultation with CECW-E.

(a) For critical structures the MDE is set equal to the maximum credible earthquake (MCE). The MCE is defined as the largest earthquake that can reasonably be generated by a specific source on the basis of seismological and geological evidence (ER 1110-2-1806). A site-specific MCE is determined by a deterministic seismic hazard analysis (DSHA). The conditions defining an intake tower as critical are described in *a* above.

(b) For other than critical structures the MDE is selected as a lesser earthquake than the MCE, which provides for an economical design meeting specified safety standards. Because the purpose of the MDE is to protect against economic losses from damage or loss of service, alternative choices of return period for the MDE may be made on the basis of economic considerations. Ordinarily, the MDE is defined for intake towers as a ground motion having a 10 percent probability of exceedance during the service life of 100 years. This results in a return period of about 1,000 years. For structures a site-specific MDE is determined by a PSHA.

(3) Damping. Damping is a naturally occurring dissipation of energy within the structure. Typical causes of damping are internal friction and hysteretic material behavior. Although damping is mainly a material and structural system-related phenomenon, it is generally incorporated within the response spectra used for analyses. Five percent of critical damping is commonly used for the analysis of intake towers for the MDE and OBE.

(4) Effective stiffness. Postyield response is appropriate when performing seismic design or evaluation for the MDE. Based on Dove (1998), during postyield response the tower will crack at the base and will form a plastic hinge, which will decrease stiffness. The reduced moment of inertia is designated as the effective moment of inertia I_E and is a function of the ratio of the nominal moment capacity M_N to the crack-in moment M_{CR} . For intake structures, base the effective moment of inertia on the following relationship:

$$\frac{I_E}{I_g} = 0.8 - 0.9 \left[\frac{M_N}{M_{CR}} - 1 \right] \quad (4-3)$$

The ratio of I_E/I_g where I_g is the gross moment of inertia (uncracked) should not be greater than 0.8, nor less than 0.35 for walls reinforced with 40 grade steel, nor less than 0.25 for walls reinforced with 60 grade steel. For the OBE the tower should be uncracked at the base; therefore, the effective stiffness is equal to the gross stiffness.

(5) Performance requirements. The performance requirements for intake structures subjected to MDE and OBE demands are provided in *e* below along with the strength requirements for both events.

(6) Earthquake directional components. Earthquake ground motion can be defined for three individual components: a principal horizontal component, a second horizontal component perpendicular to the principal horizontal component, and a vertical component. The three earthquake components should be statistically independent; therefore, the maximum structural responses due to each component will not occur at the same time. It is the designer's responsibility to coordinate with geotechnical engineers on the ground motion components required as described in EM 1110-2-6050. Methods for combining the structural responses from the two horizontal components are described in *e*(1)(d) and (e) below.

c. General design and analysis. The load conditions consist of the usual loads coupled with loads due to a horizontal component of earthquake ground motion. The vertical component of the ground motion has little influence on the overall response of a free-standing tower. Usual loads include the dead weight of the tower, hydrostatic loads due to normal pools, routine operating loads, normal debris loads, and sediment loads. The effects of the design earthquake event should be combined only with other loads that are most likely to occur coincident with normal pool conditions.

(1) Loads. The seismic analysis should account for the dynamic characteristics of the tower-water-foundation system by using the simplified two-mode, lumped-mass method, or by using finite element models. The two-mode, lumped-mass method was developed only for the preliminary design or evaluation of vertical intake structures that are surrounded by water and is not suitable for inclined towers. Finite element models should be used for the final design or evaluation of towers in seismic zones 3 and 4, or in zones 2A or 2B if the seismic load case controls the preliminary design or evaluation.

(2) Analysis. For a vertical tower, the structure may be modeled as a cantilever lumped-mass system of beam elements, and the inertial effects due to the ground motion will induce bending and shear forces in the tower. The earthquake load is modeled by a design response spectrum, or by multiple time-histories for more refined analyses. The response spectrum modal analysis provides only the absolute maximum values of forces, shears, moments, and displacements. The analysis methods and procedures, advantages and disadvantages of response spectrum modal analysis, and procedure for combining modal values are

described in Appendix B. The two-mode approximation and computer solution methods of analysis for a free-standing intake tower is described in Appendix C. A refined method of computing hydrodynamic added mass is described in Appendix D.

d. Torsional effects.

(1) General. Because of the random nature of earthquake ground motions, the direction of seismic loading must be considered. One important consequence of the direction of loading is the potential introduction of torsion. Two categories of towers, regular and irregular, are used to distinguish those towers for which torsional effects are likely to be significant. In deciding whether torsion is significant, the structural engineer should consider all factors such as access bridge effects and local irregularities in the mass or stiffness on a case-by-case basis. A designer should take all reasonable steps to provide symmetry and uniformity without detracting from the functions required for hydraulic purposes.

(2) Regular towers. Regular towers are less subject to damage than irregular towers. Regular towers have a symmetric distribution of mass and stiffness about their principal axes and along the full height. Towers are symmetric in plan if the center of mass is relatively close to the center of rigidity. If these two points are within 10 percent of the tower width when measured perpendicular to the earthquake motion, then the tower is basically symmetric. Towers that are basically symmetrical in plan throughout their height can be designed or evaluated for the bending moments and shears from two-dimensional beam model response spectra or time-history analyses. Earthquakes generate ground motions that occur simultaneously in all directions. For regular towers, the directional uncertainty of the ground motions can be evaluated by performing separate seismic analyses for each principal direction and combining the results as described in e(1)(d) below.

(3) Irregular towers. Towers that do not meet the criteria of the preceding paragraph are irregular and should be designed or evaluated for torsion. There are two different approaches for including torsion. The first method uses two-dimensional cantilever beam analyses as performed for the regular towers. On this analysis a separate calculation of torsional effects is superimposed. The torsional moment acting on a cross section is calculated as the sum of all incremental torsional moments above the cross section. An incremental torsional moment is obtained by multiplying the lateral inertial force for a given vertical portion of the tower by its eccentricity from the center of rigidity. The eccentricity is the perpendicular distance from the line of action of the inertial force to the center of rigidity. The resulting summation of the incremental torsional moments above the cross section must then be distributed to the shear-resisting elements according to their relative stiffness and superimposed with the analysis results. This method is common to building evaluations where each wall or lateral force-resisting element of the structure is required to carry both direct shear and torsional shear in accordance with its relative stiffness. A second method for computing torsional effects is to use a three-dimensional structural model to represent the mass and stiffness of the structural system. Various directions of the earthquakes can be investigated to determine the directions that govern the design. In this approach the torsional effects are automatically included in the analysis.

e. Strength and service requirements.

(1) Seismic load cases and combinations.

(a) General. Seismic design for new towers and the evaluation of existing towers must demonstrate that the tower has adequate strength, ductility, and stability to resist the specified earthquake ground motions. Flexural ductility and the benefits of energy dissipation beyond yield should be considered in the design and evaluation process for the MDE, if it can be demonstrated that the tower is not vulnerable to brittle modes of failure. Flexural ductility is indirectly accounted for in the strength equations through the use of a moment reduction factor. Intake structures should have adequate capacity (strength) to resist the MDE demand by allowing for some energy dissipation through inelastic rotation. Intake structures must

also have adequate capacity to perform elastically under OBE loading conditions. An example of seismic analysis for the determination of structural capacity is provided in Appendix C.

(b) Limitations. The provisions described herein for the seismic design and evaluation of intake towers are limited to free-standing intake towers with height-to-width aspect ratios of two or more.

(c) Strength requirements. The ultimate strength U or capacity of new and existing towers will be determined using the principles and procedures described in EM 1110-2-2104. Capacities are based on ultimate strength, or the nominal strength multiplied by a capacity reduction factor. The capacity reduction factor is 0.9 for bending and 0.85 for shear. Intake tower sections shall have the strength to resist load combinations involving dead load, live load, and earthquake load as prescribed by the following load factor equation:

For the MDE

$$U = D + L + 1.1E / R_M \quad (4-4)$$

For the OBE

$$U = 1.4 (D + L) + 1.5E \quad (4-5)$$

where:

U = value of thrusts, shears, or moments due to the effects of dead load, live load, and earthquake

D = internal forces from self-weight

L = internal forces from live loads

E = internal forces from the MDE or OBE. Two orthogonal components must be considered (see the following paragraph).

R_M = moment reduction factor (use $R_M = 1$ for shear and thrust, $R_M = 2$ for moment)

(d) Multicomponent earthquake responses. Intake towers must be capable of resisting maximum earthquake ground motions occurring in any direction. The inertial forces obtained from an analysis generally represent the effects of the principal ground motion component. For a moment, thrust, shear, or force at a particular location, the direction of the earthquake components causing the maximum value needs to be determined. Since an investigation of all possible earthquake directions is difficult, the following alternative methods for estimating peak values should be used.

(e) Circular or rectangular towers. For circular ($\alpha = 0.4$) or rectangular ($\alpha = 0.3$) towers, the orthogonal combination method should be used to account for the directional uncertainty of the MDE or OBE earthquake motions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions. This is accomplished by considering the two following combinations:

$$E = \pm[E_X + \alpha E_Y] \quad (4-6)$$

$$E = \pm[\alpha E_X + E_Y] \quad (4-7)$$

where:

E = peak positive or negative values of the forces, shears, moments, or thrust due to the alternating directional effects of the earthquake

E_X = effects resulting from the X-component of ground motion occurring in the direction of the major principal tower axis

E_Y = effects resulting from the Y-component of ground motion occurring in the direction of the minor principal tower axis.

(f) Unsymmetrical or inclined towers. The square root of sum of square (SRSS) method should be used for a preliminary evaluation of the multidirectional earthquake effects on towers that are irregular in plan and elevation. The force, shear, moment, or thrust at a particular location can be estimated from any set of orthogonal analyses by a SRSS combination.

$$E = \sqrt{E_X^2 + E_Y^2} \quad (4-8)$$

(2) Ductility.

(a) Inelastic behavior. Ductility is an indicator of inelastic deformation. Research performed by Dove (1998) shows that a reinforced concrete tower should have a displacement at failure that is equal to 150 to 350 percent of the displacement at yield. Based on a corresponding bending displacement ductility of 1.5 to 3.5, a tower should be designed or evaluated for seismic moments that are 70 to 40 percent, respectively, of the elastic moments obtained from a response spectrum or time-history analysis. Some load transfer mechanisms in reinforced concrete are brittle, and the strength may degrade after repeated cycles of loading beyond the yield level. The brittle failure modes are due to fracture of the reinforcement, an anchorage failure of the reinforcement, a splice failure (bond) of the reinforcement, a shear (diagonal tension) failure, a compressive spalling failure, and a sliding shear failure.

(b) Moment reduction factor (R_M). The moment reduction factor is the ratio of the elastic response moment to the applied seismic moment. A moment reduction factor of two should be used only when brittle failure modes cannot occur, and the structural engineer should design and detail the structure so that all the brittle failure modes are precluded. Properly detailing the reinforcement is important to assure adequate performance and to preserve the structural integrity of the tower during a seismic event. Adequate detailing of reinforcement should be provided in accordance with the provisions in (3) to (8) below. An example is in Appendix C.

(c) Confinement requirements. Some towers may require special confinement reinforcement and boundary elements. A moment reduction factor greater than two should be used only if appropriate measures are taken to confine the concrete in regions subjected to large compressive strains, and to improve the cyclic performance of splices and anchorages by providing heavy confinement reinforcement in plastic hinge regions. Plastic hinges or inelastic zones occur in regions with large moments. The confining reinforcement keeps the concrete from spalling and allows the tower to redistribute loads after cracking. Typical locations occur at the base of conduits and other openings, or at discontinuities due to abrupt changes in structural geometry. Corners should have diagonal reinforcement to control anticipated cracks. Evaluation of postelastic and moment reduction factors greater than two should be performed in consultation with and approved by CECW-EW.

(3) Fracture of reinforcement requirements.

(a) General. The tensile reinforcement can fracture suddenly if the reinforcement is less than 1 percent and cause a brittle failure. To prevent a brittle failure mechanism, the nominal moment capacity should equal or exceed the uncracked moment capacity by at least 20 percent. The cracking moment can be calculated using Equation 4-9:

$$M_{cr} = \left(\frac{I_g}{C} \right) \left(\frac{P}{A_g} + f_r \right) \quad (4-9)$$

where

C = the distance from the neutral axis to the extreme fiber

P = axial load on the tower (positive for tension)

A_g = gross section area (uncracked)

f_r = modulus of rupture, $0.62\sqrt{f'_c}$ (MPa units), $7.5\sqrt{f'_c}$ (psi units) where f'_c is the concrete compression strength

(b) Existing towers. A force-based or a displacement-based approach may be used to evaluate the minimum reinforcement requirement if the nominal moment capacity is less than 120 percent of the cracking moment.

(c) New towers. For all seismic designs, sufficient reinforcing steel should be provided to assure that the nominal moment capacity equals or exceeds 120 percent of the cracking moment (Equation 4-10):

$$M_N \geq 1.2M_{cr} \quad (4-10)$$

(4) Anchorage failure mode.

(a) Existing towers. The flexural strength of the intake tower will deteriorate during a major earthquake if the vertical reinforcement is not adequately anchored. For straight bars, the anchorage lengths should be greater than required by Equation 4-11:

$$l_a = \frac{2.626k_s d_b}{\sqrt{f'_c} \left(1 + 2.5 \frac{c}{d_b} \right)} \quad (\text{KPa units}) \quad (4-11)$$

$$l_a = \frac{k_s d_b}{\sqrt{f'_c} \left(1 + 2.5 \frac{2}{d_b} \right)} \quad (\text{psi units})$$

where

l_a = minimum required effective anchorage length of longitudinal reinforcement

k_s = a constant for reinforcing steel with a yield stress of f_y

$$k_s = \frac{(f_y - 76845)}{33.1} \quad (\text{KPa units})$$

$$k_s = \frac{(f_y - 11000)}{4.8} \quad (\text{psi units})$$

f_y = yield stress of the longitudinal reinforcement

d_b = nominal bar diameter

c = the lesser of the clear cover over the bars, or half the clear spacing between adjacent bars, in.

The term $2.5(c/d_b)$ introduces the effect of clear cover to the calculation of development length. The value of (c/d_b) shall not be greater than 2.5. For 90-degree standard hooks, the anchorage provided should be greater than that required by Equation 4-12. For anchorage with 90-degree standard hooks, the effective anchorage length in inches is

$$l_a = 1,200d_b \frac{2.626f_y}{60,000\sqrt{f'_c}} \quad (\text{KPa units}) \quad (4-12)$$

$$l_a = 1,200d_b \frac{f_y}{60,000\sqrt{f'_c}} \quad (\text{psi units})$$

(b) New towers. The required effective anchorage length of longitudinal reinforcement should be equal to or greater than that provided by Equations 4-11 and 4-12. The minimum anchorage length, however, should not be less than 30 bar diameters for straight anchorage or less than 15 bar diameters for hooked anchorages.

(5) Splice failure of bending reinforcement.

(a) Existing towers. Splices in the flexural steel, if located in a plastic hinge region, may undergo strength deterioration during a major earthquake. When compressive strains in the concrete are less than 0.002 in./in., the splices will perform satisfactorily without transverse confinement steel. When concrete compressive strains exceed 0.002 in./in., transverse confinement steel should be provided at splice locations. The minimum area of transverse reinforcement A_{tr} necessary to prevent bond deterioration is:

$$A_{tr} = \frac{sf_y}{l_s f_{yt}} A_b \quad (4-13)$$

where

A_{tr} = minimum area of transverse reinforcement

s = average spacing of transverse reinforcement over the splice length

l_s = splice length

f_{yt} = yield stress of the transverse reinforcement

A_b = area of the spliced bar

(b) New towers. Deterioration of bond and splice strengths of reinforcing bars is a critical potential failure mode for reinforced concrete structures. Transverse reinforcement provides the best protection against splice strength degradation. Perimeter transverse reinforcing steel in the amount equal to or greater than that indicated by Equation 4-13 should be provided at all splice locations where concrete compressive strains are expected to exceed 0.002 in./in. Transverse steel provided to resist shear and bending forces induced by hydrostatic or gravity loads may also be considered as effective transverse confinement steel and used to prevent splice failures. Perimeter transverse confinement reinforcement using smaller bars at close spacings is better than that obtained using larger bars at wide spacing.

(c) Splice length requirements. Splice performance will be greatly improved if splices are located away from potential plastic hinge regions and if lap splice locations are staggered (i.e., no more than half of the bars spliced at any horizontal plane). The lap splice length required for new and existing towers should not be less than $154 d_b \div \sqrt{f'_c}$ MPa units ($1,860 d_b \div \sqrt{f'_c}$ psi units). Existing towers with lesser splice lengths may perform acceptably if splices are staggered. Lesser splice lengths should be evaluated according to their location and seismic demand.

(6) Shear (diagonal tension) failure. For existing and new towers, the capacity of the intake tower in shear shall be equal to or greater than the lesser of the full elastic demand placed on the tower by the design earthquake, or the shear corresponding to 1.5 times the shear associated with the nominal flexural strength of the tower. The capacity of the concrete in shear and the shear resistance available from the transverse reinforcing after deductions for other live load effects may be considered. The shear capacity of the tower includes contributions from both the concrete and the shear reinforcement. The total ultimate shear strength V_U is

$$V_U = \theta (V_C + V_S) = 0.85 (V_C + V_S) \quad (4-14)$$

where V_C is the contribution from the concrete:

$$V_c = 2 \left[K + \frac{P}{13.8 A_g} \right] 0.083 \sqrt{f'_{CA} \cdot A_E} \quad (\text{KPa units}) \quad (4-15)$$

$$V_c = 2 \left[K + \frac{P}{2,000 A_g} \right] \sqrt{f'_{CA} \cdot A_E} \quad (\text{psi units})$$

where

$K = 1$ for $R_M = 1$; $K = 0.5$ for $R_M = 2$

P = axial load on the section

f_{CA} = actual concrete compression strength (typically $f_{CA} \geq 1.5f_c$)

$A_E = 0.8 A_{gross}$ (A_{gross} = actual cross-section area)

and V_s is the contribution from the shear reinforcement as defined for circular towers as equal to:

(a) For circular towers:

$$V_s = \frac{\pi A_h (f_y) (0.8d)}{2s} \quad (4-16)$$

where

A_h = horizontal reinforcement cross-section area

d = outside diameter of the tower

f_y = stress in the transverse reinforcing due to unfactored hydrostatic loading and other non-earthquake loads

s = spacing of reinforcement

(b) For rectangular towers,

$$V_s = \frac{A_h (f_y) (0.8d)}{s} \quad (4-17)$$

where d is the tower dimension in the direction of the seismic shear force. Since this method uses an interaction equation to evaluate biaxial shear (see Appendix C), the earthquake forces must be in the directions of the principal axes.

(7) Compressive spalling failure. In existing and new towers, excessive compressive strains can cause spalling of the concrete cover and rapid degradation of the transverse confining reinforcement, which can lead to splice failures and buckling of the longitudinal reinforcing steel. However, the chance of compressive spalling failures is small because the compressive stresses and the reinforcing steel percentages are low in intake towers. Compressive spalling failures will not occur at ultimate load conditions if the concrete compressive strains are less than 0.4 percent, or if the location of the neutral axis is less than 15 percent of the effective depth to the centroid of the reinforcement. The latter condition can be expressed as

$$\frac{c}{d} \leq 0.15 \quad (4-18)$$

where

c = distance from extreme compression fiber to the neutral axis, in.

d = distance from extreme compression fiber to the centroid of tension reinforcement, in.

(8) Sliding shear failure investigation. In existing and new towers, the potential for sliding along a horizontal crack at all possible failure planes within the structure should be evaluated. The nominal sliding

shear strength can be used to determine if a sliding shear failure is possible. The nominal shear strength can be determined using Equation 4-19:

$$V_{SL} = P + 0.25 f_y A_{VF} \quad (4-19)$$

(Assuming coefficient of friction $\mu = 1$)

where

V_{SL} = nominal sliding shear strength

P = axial load on section

f_y = yield strength of reinforcement

A_{VF} = area of shear friction reinforcement (area of vertical reinforcement crossing the horizontal failure plane)

The sliding shear resistance in KN (kips) calculated by use of Equation 4-19 shall not be greater than

$$0.0008 \sqrt{f'_c} A_g \text{ MPa} \quad (0.01 \sqrt{f'_c} A_g \text{ psi})$$

where A_g is the gross area of concrete normal to the horizontal failure plane. It should be noted that sliding shear does not necessarily constitute a failure condition, since the permanent displacements that occur during sliding may not be sufficient to cause structural instability.

(9) Displacement-based analysis of existing rectangular and circular towers.

(a) Tower performance. Many existing intake towers are lightly reinforced structures (steel percentage less than 0.50 percent) that may have a nominal moment capacity less than 120 percent of the cracking moment. If the nominal moment capacity exceeds the cracking moment, then the tower will be able to form a plastic hinge at critical points of structural damage similar to the behavior of reinforced concrete bridge piers or building columns. Towers that do not meet this minimum requirement will have plastic hinge lengths that are much less than those in buildings or bridges, and these towers are more likely to fail by fracturing the longitudinal reinforcement. Recent analytical and experimental research that addressed the nonlinear seismic response and ductility of lightly reinforced towers has shown that many towers will have limited ductility corresponding to a highly localized failure of the reinforcement within a single crack (Dove 1998). The critical crack may form at the base of the tower or at other abrupt changes in structural stiffness. Typical rectangular or circular towers will have small plastic hinge lengths with a limited displacement capacity.

(b) Displacement demand. A displacement-based dynamic analysis may be used to evaluate this localized failure mode, and to determine if the ultimate displacement capacity δ_U at the top of the tower (as shown in Figure 4-1) exceeds the

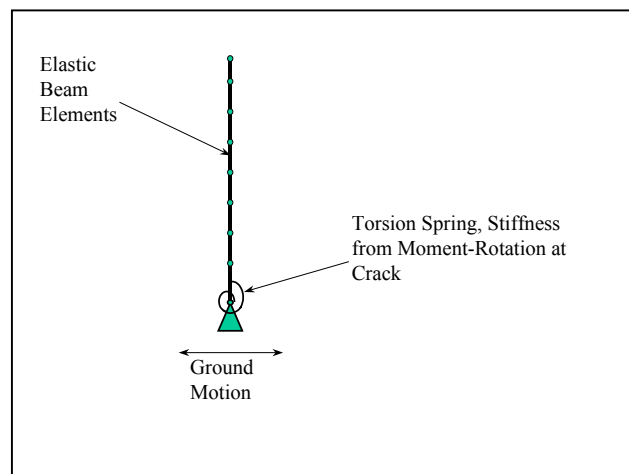


Figure 4-1. Tower model of displacement demand

displacement demand δ_D for the MDE. This technique is a modified response spectrum analysis that considers the earthquake-induced displacements of a tower, and uses effective stiffness properties ($b(4)$ above) to account for the shift in the fundamental frequencies of the tower due to formation of plastic regions. The displacement demand is the maximum deflection calculated at the top of the tower from a response spectrum analysis using a linear spring stiffness, the beam element properties, and the added mass due to surrounding or contained water. The tower is modeled as a cantilever beam that is supported at the base on a rotational spring (Figure 4-1). The beam represents the elastic response of the tower above the crack, and the spring represents the nonlinear response of the cracked region. The nonlinear rotational stiffness of the spring may be estimated by using an appropriate reinforced concrete section analysis computer program to calculate the moment-curvature relationship for the cross-section of the tower, and then multiplying the curvature values by an appropriate plastic hinge length to plot the corresponding moment-rotation relationship. The moment-rotation relationship represents the stiffness of the rotational spring, and this relationship is often strongly bilinear. The following simplification is required to approximate the spring stiffness as a linear relationship in the elastic response spectrum analysis. First, calculate the area under the bilinear moment-rotation curve from the origin to the value of the maximum allowable rotation, and then generate a fictitious linear, moment-rotation curve that has the same area and maximum rotation. The fictitious spring stiffness should have the same total energy and maximum rotation as the bilinear curve, but the rotational stiffness and the frequency of spring response will lie between the real elastic and inelastic values.

(c) Displacement capacity. The displacement capacity at the top of the tower is related to the height, the length of the plastic hinge, and the fracture strain capacity (approximately 0.05 in./in.) of the reinforcement. The displacement capacity may be modeled as shown in Figure 4-2 and discussed below.

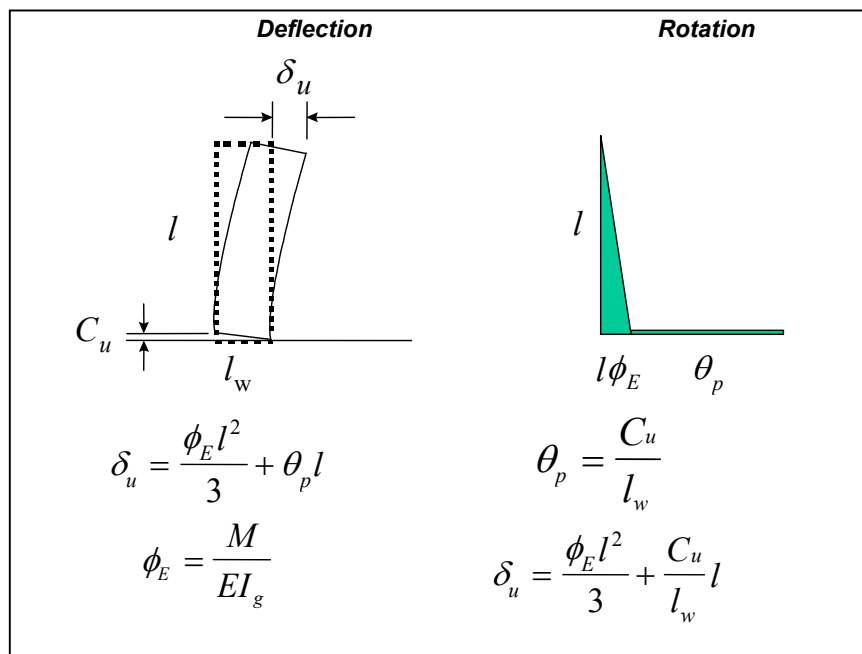


Figure 4-2. Tower model of displacement capacity

where

δ_u = the ultimate displacement capacity

ϕ_E = the elastic curvature at cracking (at the base of the tower)

l = the height of the tower above the crack

θ_p = the plastic rotation at failure

M = the moment at the base of the tower

C_u = the ultimate crack width at failure

l_w = the depth of the section

The ultimate lateral deflection is modeled as the sum of an elastic response of the tower above the cracked section and a rigid body rotation of the tower as the crack opens at the base. It is conservative to assume the tower rotates about a neutral axis of the cracked section that is coincident with the edge of the tower. Hence, the lateral rigid body deflection at the top of the tower varies directly with the crack width, and its maximum value varies as a ratio of the tower height to the tower width, times the ultimate crack width. The ultimate crack width at failure C_u is a function of the ultimate strain in the reinforcement and the strain penetration. Recent experiments have modeled the cyclic response of single bars for different concrete strengths, bar sizes, and bar strengths. Statistical analyses of these experiments indicate that for a single crack response, the crack widths are reliably predicted by the ultimate strain capacity of the bar and the bar diameter. An appropriate value of the ultimate crack width may be estimated using the following empirical equation:

$$C_u = 0.12 + 2.47\varepsilon_u + 0.312d_b \quad (4-20)$$

where

ε_u = ultimate strain at failure of the bar as measured over a standard 20.32-cm (8-in.) gage length

d_b = diameter of the reinforcing, cm

(d) Tower example. A displacement-based evaluation of the tower in Appendix C is based on a failure mechanism that may be characterized by a single crack at the base of the tower. The analysis is summarized below.

(1) Compute the moment-curvature relationship for the bottom section of the tower about the weak and strong axes, and include the deadweight of the tower. For an 18 percent ultimate strain, the ultimate crack width and the strain penetration length L_s may be estimated as 1.65 cm (0.65 in.) and approximately 10.0 cm (4.0 in.), respectively, from the previous empirical equation and the following equation:

$$L_s = C_u / \varepsilon_u \quad (4-21)$$

(2) Multiply the moment-curvature values by this strain penetration length to obtain the moment-rotation relationship (Figure 4-3). The values of the linear spring stiffness for bending about the weak and strong axes are estimated based on having equal total areas under the actual and fictitious moment-rotation curves. The rotational spring constants for the weak and strong axes are 2.107E+12 N-m/Radian and 3.561E+12 N-m/Radian, respectively.

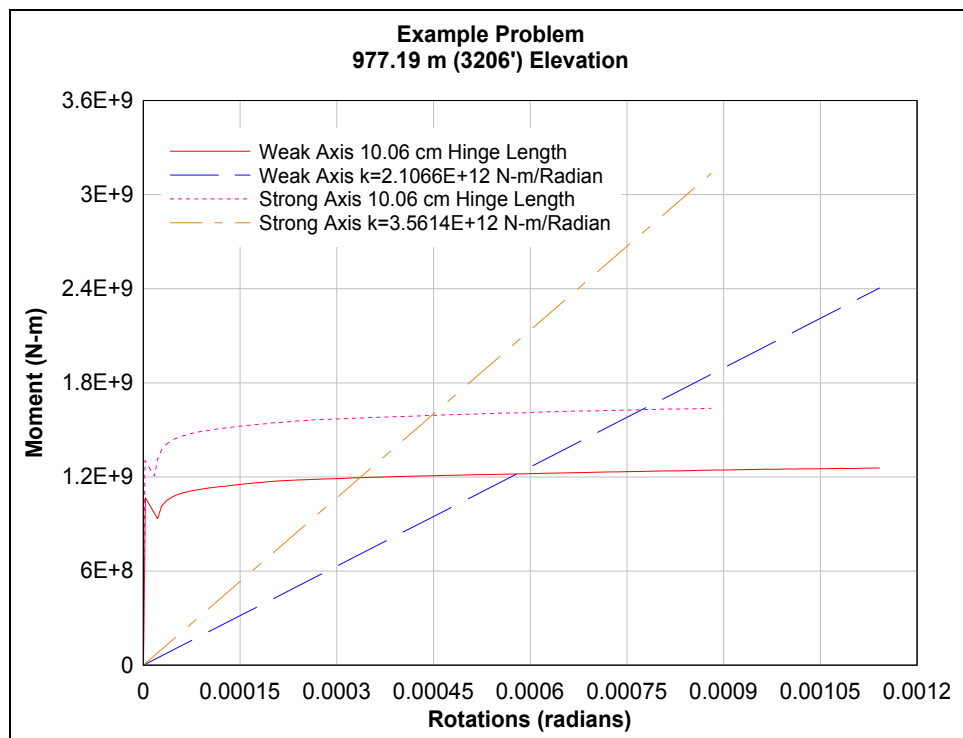


Figure 4-3. Moment-rotation relationship for the tower base

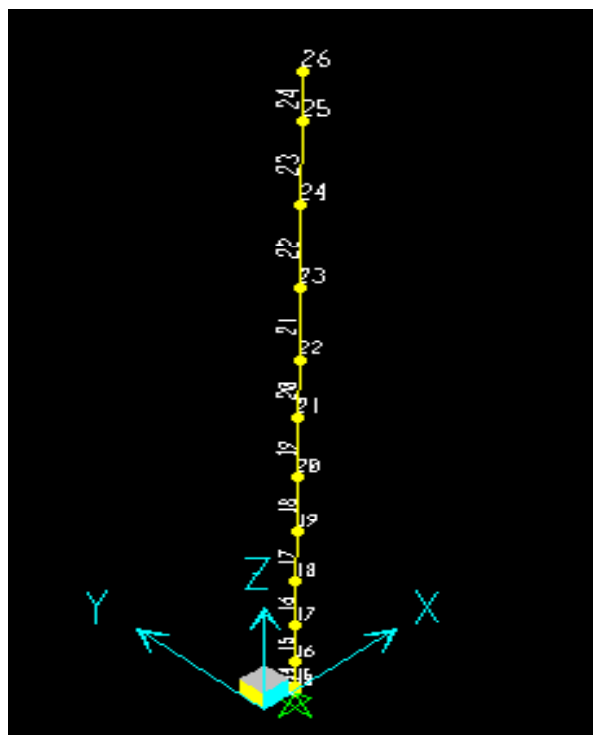


Figure 4-4. SAP 2000 intake tower model

(3) Perform a structural analysis for the MDE response spectrum with 5 percent damping (as shown in Figure C-3) using commercial software such as SAP 2000. This is similar to the analysis presented in Section C-8 except for the hinge supports and rotational springs provided at the base of the tower to model the properties of the cracked section (Figure 4-4). The same section properties and masses were used (as shown in Figures C-1 and C-5) to model the tower and water. The sectional properties of the tower were uniformly distributed, and the added masses ($m_a^o + m_a^i$) were lumped at nodes only (as shown in Tables C-3 and C-4). The deflections calculated at the top of the tower are 9.6 cm (3.8 in.) and 10.1 cm (4.0 in.) for the strong and weak axes, respectively.

(4) The elastic curvature ϕ_E at the base is read from the moment-curvature relationship for the section. The ultimate base rotation and the ultimate displacement capacity are calculated using the following equations:

$$\theta_p = C_u / l_w \quad (4-22)$$

$$\delta_u = \frac{\phi_e l^2}{3} + \theta_p l \quad (4-23)$$

(5) The deflection capacities at the top of the tower are 10.7 cm (4.2 in.) and 14.2 cm (5.6 in.) about the strong and weak axes, respectively. Therefore the tower satisfies the performance requirements for the MDE.

4-7. Special Guidance for Inclined Intake Towers

a. Inclined and vertical towers supported along one side by the rock abutment will experience complicated bending, shear, and torsion under earthquake ground shaking. The ground motions excite the tower not only from the base but also along the entire abutment support. The three-dimensional geometry of supported towers combined with composite seismic input requires a three-dimensional finite element modeling of the tower for earthquake response analysis. Generally, supported towers should be discretized using a combination of three-dimensional solid and shell elements, as appropriate, or simply by three-dimensional solid elements. However, some components of the tower such as the trashrack can be modeled more appropriately using beam elements. The interaction with the foundation and abutment may be considered by including a portion of the foundation and abutment regions as part of the tower model. The foundation and abutment models should be developed to have dimensions comparable to those of the tower section. Competent rock sites may adequately be represented by a massless foundation model while foundation soil models are usually represented by equivalent springs, by a combination of springs and dashpots, or by finite element soil models. More detailed guidance on foundation-structure interaction is covered in EM 1110-2-6050 and EM 1110-2-6051.

b. The effect of hydrodynamic pressures on supported towers is represented by the equivalent added-mass concept. The added hydrodynamic mass, however, depends not only on the geometry of the tower but also on topography of the surrounding abutment-foundation region. If the effect of topography on the added mass is judged significant, the added mass should be computed using the finite element or boundary element procedures as discussed in EM 1110-2-6051.

c. The dynamic response of a supported tower is a complex combination of biaxial bending and torsion depending on the tower-abutment-foundation and tower-water interaction. This condition is best treated by three-dimensional response-spectrum (EM 1110-2-6050) and time-history analyses (EM 1110-2-6051). The analysis may start with the response-spectrum modal superposition followed by time-history procedure. The seismic input consists of three components of ground motions in the form of response spectra or acceleration time-histories applied at the fixed exterior nodes of the foundation model.

A response-spectrum analysis provides only maximum stresses and section forces. A time-history analysis provides both the maximum magnitudes and distributions of stresses and section forces at any time during a seismic event. In case of severe seismic events, both magnitudes and time variation of stresses and forces may be required to design or evaluate the structure and the anchorage system.

4-8. Remedial Strengthening of Existing Towers

The seismic analysis indicates an existing intake tower is in need of a seismic retrofit, or when nonlinear, inelastic seismic analysis procedures are necessary to investigate performance of the structure. The need for remedial strengthening will be determined on a case-by-case basis after suitable ductility evaluation has been performed. Any evaluations of existing intake towers that indicate the need for remedial strengthening shall be done in consultations with CECW-EW.

Chapter 5

Gates, Valves, Bulkheads and Guides, Trashracks, and Operating Equipment

5-1. Functions and Requirements

Gates, valves, and bulkheads in intake structures serve a number of different functions related to the purposes of the project. Type, location, arrangement, layout, and design will depend on the various operational, maintenance and servicing, dam safety, and construction (if used for diversion) requirements. Further discussion and details on gates and valves are provided in *Advanced Dam Engineering for Design Construction and Rehabilitation* by Robert Jansen and EM 1110-2-1602, Chapter 3.

a. Low-level drawdown intakes. Low-level drawdown intakes are required on new projects to lower the reservoir below minimum pool elevation or to provide a dry reservoir behind the dam. The reasons for this capability may include provisions to draw down the reservoir for inspection after a seismic event, for periodic inspections and repair of features or structures normally submerged, to flush fine sediment accumulations in the reservoir, to provide flexibility for changes in reservoir regulation, and to provide no pool during low flows to meet environmental requirements. Whether these intakes are gated or bulkheaded will depend on economics, frequency of use, and the type of operation. The criteria for low-level drawdown are addressed in ER 1110-2-50.

b. Control gates. The control gates located in each gated passage to the outlet tunnel or conduit generally consist of an emergency gate followed by a service gate. Guides are provided just upstream of the gates for stoplogs or a bulkhead. The number of gates and passageways, size, types, and layout in relation to the structure will depend on the project purpose(s); the hydraulic design, operation, and flow regulation requirements; hydraulic head; and structural design, construction, and economic requirements. Hydraulic criteria for the design of gates are discussed in EM 1110-2-1602.

(1) Service gates. Service gates are used for flow regulation, and one gate is required for each water passage. For low- and medium-head conditions, the type of gate may be a tractor (mechanically or hydraulically operated), hydraulically operated slide gate, or hydraulically operated tainter gate. For high-head conditions, hydraulically operated slide gates are generally preferred for long periods of operation at partial gate openings.

(2) Emergency gates. Emergency gates should be located within the intake structure immediately upstream of the service gates. Emergency gates are required in reservoirs having water conservation, power, or a similar type pool to prevent loss of stored water if a service gate is inoperable (see EM 1110-2-1602, Chapter 3, Section III, Gate Passage Requirements for additional guidance on emergency gate requirements). Emergency gates must be designed to close under free flow at the maximum conservation or power pool. However, emergency gates are not required to regulate flow, as a service gate, because sufficient conservation or power releases can be made through the remaining facilities. Typically, emergency gates are of the same type as the service gate; however, for low- and medium-head conditions, a single transferable tractor gate installed by a traveling hoist at the operating deck near the top of the structure may be used. Emergency gates must be designed to withstand the maximum reservoir head at stresses not exceeding 33-1/3 percent of the basic stress.

c. Water temperature and quality. Projects providing temperature or water quality controls may have multi-level intake ports or telescoping weirs with a mixing wet well. The intakes may be gated or ungated. Flow regulation with control gates is required downstream of the wet well.

5-2. Gate Types

A wide variety of gates have been used in outlet works. The principal types of regulating gates on U.S. Army Corps of Engineers dams for heads of 100 ft or more are hydraulic slide gates, caterpillar gates with cable hoists, hydraulic caterpillar or wheel gates, and hydraulic tainter gates. The type selected depends on the purpose, operating characteristics, flow regulation requirements, maintenance and serviceability, and life-cycle cost.

a. Hydraulically operated slide gates. The hydraulically operated slide gate has a record of dependability and low maintenance and is capable of providing fine regulation. These gates have been designed for projects up to approximately a 500-ft head. Compared to the tractor or roller gates and tainter gates, hydraulically operated slide gates for high heads are smaller because of the much greater operating force per square foot of gate and the greater rigidity required to meet the operating requirements. The gate is housed in steel gate frames with attached conduit liners requiring close fabrication tolerances to provide a watertight unit. The assembly is embedded and anchored directly in the main concrete body. The seating or sealing surfaces of embedded gate frames must be accessible for inspection and maintenance. Cavitation problems have occurred in the gates' leaf bottom edges and the gate frame downstream of the gate under high-velocity flow conditions at small gate openings.

b. Roller- and wheel-mounted gates. Roller-mounted (tractor or caterpillar) gates have roller trains mounted on the gate parallel to the gate slot, and wheel-mounted gates use fixed wheels attached to the gate. Both gate types can be hydraulically operated or are suspended with cable hoists and hydraulically operated gates for low-flow regulation. The cable hoist gates are not as positive for low-flow regulation as other gates because of the elastic properties of the cables. These types of gates are used for heads up to about 200 ft. A gate frame and conduit liner are provided with the gate. The roller trains and wheels on the gates are vulnerable to seizing caused by corrosion and debris.

c. Tainter (radial) gates. Radial gates are simple, reliable, and generally less expensive than other gates of equal size. They are satisfactory for low-flow regulation. Because gate slots are not required, head loss is minimized. Top seals may limit heads to about 250 ft. Eccentric trunnion designs have been used to facilitate sealing. In some cases, vibration has resulted where bottom leaf seals and beams were not properly designed.

5-3. Bulkheads and Guides

A bulkhead or stoplog and guides should be provided at the entrance to each water passage for dewatering the service and emergency gates for maintenance except when they are excessively costly or clearly unfeasible and other means can be improvised. When emergency gates are not required, stoplog or bulkhead guides for maintenance should always be provided upstream of the service gates.

5-4. Trashracks

Trashracks are needed to prevent clogging and debris damage to outlet control gates and turbines. The trash-rack type and size of openings depend on the pool elevation, intake elevation, the size of the outlet conduit, the reservoir trash conditions, type of control device used, use of the water, and the need to exclude the trash.

a. Intakes for flow regulation. A simple trash structure, usually of reinforced concrete, with clear horizontal and vertical openings not more than two-thirds the gate width should be provided at the upstream end of the outlet works to catch trees and other large trash which may reach the entrance and be capable of blocking the gated passages (EM 1110-2-1602, Chapter 3, Section II). Large trash at the tunnel entrance occurs more often when the permanent pool is only slightly above the entrance than when the permanent pool

is high above the entrance. Only in special cases in which trees and floating debris are absent from the reservoir and watershed should the trash structure be omitted.

(1) Metal trashracks fabricated of closely spaced bars are generally used for small conduits with control valves and water supply intakes that require screening of small debris.

(2) To facilitate trash removal immediately after a flood, the working platform at the top for raking and removal of trash usually should not be lower than the top elevation of the conservation or maximum power pool. The usual trash structure consists of upright beams, inclined slightly downstream from the vertical to facilitate raking, and horizontal beams.

(3) If the gate structure is located at the upstream end of the outlet tunnel, the trash structure is combined with it for economy.

(4) The area of the openings in the trash structure should limit the local net-area velocity to less than 10 to 15 ft/sec.

b. Power intakes. EM 1110-2-3001 discusses the requirements for power intake trashracks and the design loads and spacing of trashracks from the standpoint of hydraulic and structural requirements.

c. Floating trash and debris control facilities. Floating trash and debris control can be provided by a basic trash boom constructed of logs or floating pontoons. Trashracks would still be necessary to protect intakes from occasional water-logged trees or large logs that pass under the trash boom.

5-5. Load Cases

a. Gates, valves, bulkhead, and stoplogs. The gates, valves, bulkhead, and stoplog design loads should include dead loads, static live loads, dynamic live loads, temperature loads, and any unusual load which can potentially occur due to misoperation. Dead loads include self weight and the weight of the attached equipment. The static live load includes interior and exterior water surface pressures, silt pressures, temperatures, debris, ice, lifting and installation forces on components, and any other loading pertinent to the project operation. Dynamic loads include seismic, dynamic hydraulic conditions, load rejection for power intakes, vibration, fluctuating hydraulic forces, waves, and hydraulic drawdown forces for closure under flow conditions.

b. Trashracks. The trashrack structure should be designed to withstand hydraulic pressure based on velocity head loss with a minimum design head of 5 ft. External forces resulting from reservoir ice, debris clogging, and sediment buildup should be considered in the design at projects where those conditions apply.

5-6. Strength and Serviceability Requirements

Strength and serviceability requirements have an important bearing on the development and design of gates, valves, bulkhead and guides, trashracks, and operating equipment. The types of factors that should be considered include frequency and intensity of operation, cavitation and abrasion during operation, maximum deflection for operation, vibration and hydrodynamic loading, corrosion, and maintainability. Gates, bulkheads, stoplogs, trashracks, and other steel features should be designed in accordance with the strength and serviceability requirements presented in EM 1110-2-2105.

5-7. Gate Room Layout/Operating Equipment

a. Gate room layout. The gate room layout will depend on the mechanical equipment and electrical controls which will be located in the room. Adequate room size must be allowed for safe clearance and maintainability of operating equipment such as gate operators, valves, and electrical components. The room must also have adequate clearance for removal of the gates and valves. Vertical intake structures generally utilize a crane on the intake deck to disassemble and remove items requiring major maintenance. This work requires that hatches be provided to facilitate removal of components. Inclined intake structures may require a bridge crane in the gate room for disassembly of components. Lifting eyes should be strategically placed in the gate room along with provisions for monorail hoists, if necessary.

b. Operating equipment. Additional information about gates and valves is provided in EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works; EM 1110-2-2702, Design of Spillway Tainter Gates, and UFGS 11287A, Tainter Gates and Anchorages.

(1) Slide gates. Slide gates are typically hydraulic cylinder operated. The hydraulic power units can be package units. Two pump/motor combinations are used, with the hydraulic piping valved in such a way that one pump may be removed from service without rendering the system inoperative. A mechanical position indicator can be used to show the gate opening. This position indication is converted into an electronic signal for remote sensing and operation. Figure 5-1 shows a typical gate room layout with slide gates.

(2) Roller gates. Roller gates can be operated by the use of hydraulic cylinders or wire rope hoists. Position indication can be noted by use of an indicator rod or, in the case of hoists, by use of a revolution counter.

(3) Tainter gates. Tainter gates are typically hoist operated. A single motor/reducer drives a torque tube which drives a wire rope drum on each end of the tainter gate.

(4) Valves. Valves are generally electric motors operated from a pushbutton station, both locally at the valve and at a control in the gate room. Types of valves include ball valves, which act as a guard valve to fixed-cone valves in a minimum discharge type outlet, and gate valves, which are used in a main outlet conduit fill pipe.

(5) Controls and instrumentation. Controls and instrumentation for gate or valve positioning, emergency generator, and sump and utility pumps may be local, remote, or both. Piezometers, flow meters, and level gauges are discussed in EM 1110-2-4205, Hydroelectric Power Plants Mechanical Design.

c. Other equipment. Heating and ventilating of the gate room are generally achieved through the use of a ventilation fan, duct work, and unit heaters located strategically in the gate room. Minimum outdoor air quantities should be required in accordance with American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE) criteria. Special design features may be required in situations where the possibility of infiltration of gases from the decay of vegetation into the tower exists. Emergency power is usually provided by an emergency generator located at the top of the intake tower. However, in towers, emergency power may be provided by an emergency generator located in a remote building away from the intake tower. Sump pumps are normally required at the lowest point of the tower. When sump pumps are required, the discharge is pumped into the reservoir through an oil/water separator. Overhead cranes (bridge type or monorail) may be included to facilitate maintenance functions. The coverage envelope should be large enough to permit access to all gates and valves.

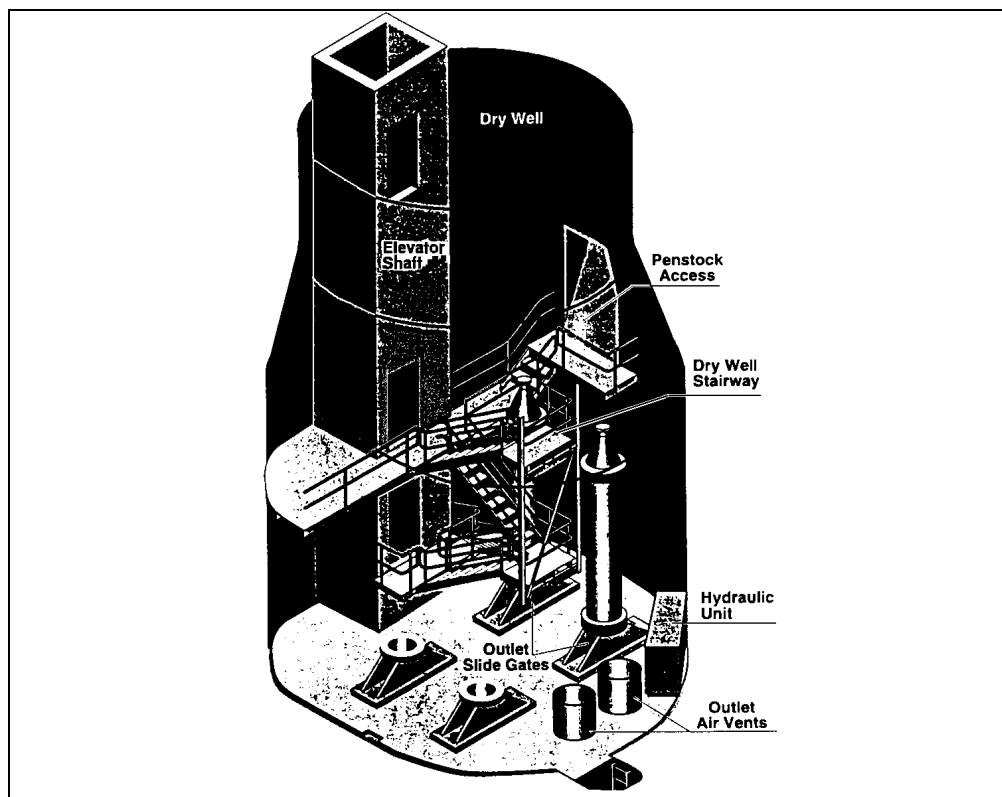


Figure 5-1. Gate room perspective

5-8. Elevator

Design and justification factors for elevators are discussed in EM 1110-2-4205, Hydroelectric Power Plants Mechanical Design. UFGS 14210A, Elevators, Electric, should be in accordance with American Society of Mechanical Engineers (ASME) A17.1, Safety Code for Elevators and Escalators.

Chapter 6

Special Detailing Requirements and Other Design Considerations

6-1. Introduction

After the general arrangement of the intake structure has been determined, access, openings, and operational and maintenance provisions should be developed to provide a functional project.

6-2. Access Bridge Requirements

a. General. Where necessary, a service bridge extends from the top of the dam or from the valley wall to the intake tower to provide access for the operation, repair, and maintenance of the gates, equipment, and appurtenances. A single-lane access bridge is sufficient. The bridge is designed for the AASHTO standard truck loading and for all possible off-highway loads due to cranes lifting or transporting gates or other equipment into or out of the intake tower. The designer should thoroughly investigate the methods used to transport and remove or install gates to make sure that all possible bridge loadings have been accounted for in the design. Generally, a single-span bridge is most economical due to the high costs of constructing intermediate supporting piers. The single spans may be prestressed girders, rolled steel beams, welded plate girders, or steel trusses. Prestressed concrete girders and rolled steel girders are economical for spans up to 110 ft. Welded plate composite steel girder bridges are commonly used for spans from 110 to 175 ft and have been used for spans up to 350 ft. However, steel trusses or intermediate piers should be considered when spans exceed 175 ft. The engineer should determine whether it is best to accommodate expansion at the abutment end or tower end of the bridge. Attaching the bridge at the tower with a fixed bearing prevents the tower and bridge from hammering during an earthquake. All bearings should have adequate seat widths and restraints to prevent loss of support during an earthquake. When the bridge is attached to the intake tower with a fixed bearing, the bridge and tower should be combined in a single analytical model to determine the combined effects of the bridge tower system when subjected to the design earthquake ground motions. The bridge superstructure will normally be capable of withstanding all the seismic inertial forces without difficulty. The objectives of the seismic analysis are to ensure that the bridge bearings and supports can accommodate the seismic inertial forces and displacements and that the supporting piers, abutments, and their foundations can accommodate the seismic inertial forces.

b. Load criteria. The bridge should be designed to withstand safely all group combinations of forces in AASHTO that are applicable. The bridge should be designed by load factor design methods using the appropriate load factors and strength reduction factors in AASHTO. The design for earthquake loadings should employ the response spectrum methodology with the earthquake demands in accordance with those established for the dam and intake tower. Impact may be omitted except when a mobile crane or gantry crane is used to pick up a gate from the bridge. When it is anticipated that construction equipment will have to use the bridge, the bridge shall be designed to accommodate construction load conditions.

6-3. Tower Service Deck Requirements

a. General. Simplicity and economy are the primary considerations in design of the intake tower service deck. Although the intake structure is essentially a reinforced concrete structure, economy may be affected and construction simplified by the use of steel beams in the operating and service decks, steel roof joists, etc.

b. Load criteria.

(1) The service deck at the top of the intake structure should be designed to withstand 250 lb/ft², or a loaded crane or truck, whichever controls. The removable hatch covers and openings should be protected by a structural steel frame. All edges exposed to traffic and heavy equipment should have armor angle protection.

(2) Intake structures with gate hoisting equipment on the operating deck are designed for the loads of the gate hoists with maximum gate operating loads. The floor slab between hoists should be designed for a uniform load of 250 lb/ft² unless a different load is warranted. The service deck should be designed for the concentrated dead load of a service gate in each service gate opening or the emergency gate in any emergency gate opening and supported by logs or blocking at the edge of the opening. The house above the operating deck is designed according to usual load and stress criteria for buildings unless a different load is warranted.

6-4. Concrete Temperature Control Measures

Cracking occurs when tensile stresses within the concrete exceed the tensile strain capacity of the concrete. In mass concrete, primary stresses result from volume changes that occur as the concrete cools from its initial peak temperature down to ambient air temperature. If not controlled or limited to acceptable levels, the tensile stresses will quickly exceed tensile strain capacity of the concrete and result in unacceptable cracking. Measures can be taken to prevent the tensile stresses from exceeding acceptable values only after the tensile strain capacity of the concrete has been determined. To achieve proper temperature control, the designer must determine various thermal properties for both the aggregates and the concrete. These properties include thermal conductivity, diffusivity, specific heat of hydration, coefficient of thermal expansion, etc. After the tensile strain capacity of the concrete and the amount of thermal contraction that can occur without causing cracking have been determined, specific measures can be taken to limit the initial peak temperature. One measure is to reduce concrete placement temperatures, and thereby lower final peak concrete temperatures. Other measures can be taken (individually or in combination) that will be successful in controlling temperature gains sufficiently to reduce or prevent cracking. Some of these measures are as follows: reducing Portland cement contents, use of fly ash as Portland cement replacement, use of chilled water or ice to reduce initial concrete placement temperature, provision of postcooling techniques such as using cooling coils, cooling aggregate stockpiles, using the largest size aggregate practical, placement of concrete in thinner lifts, and/or extending intervals between concrete placements. For additional discussion on crack control, see ACI 224R-90 Committee Report, and ETL 1110-2-365, "Nonlinear Incremental Structural Analysis of Massive Concrete Structures."

6-5. Air Vents

Circular or rectangular air vents are placed at the downstream side of the service gates to reduce cavitation damage and prevent erratic flow conditions. Vents are extended above pool elevations, preferably above the spillway design flood pool, and are arranged to open outside the structure far enough away from the service bridge or any platform where personnel could possibly be endangered by the strong inflow of air during outlet operation. When convenient and economical, the air vents from two or more gates may be brought together at a point above the pressure gradient into a larger vent. Vents may be steel pipes or may be formed in concrete. Appropriate air venting should also be provided for dewatering and filling operations downstream of valves, bulkheads, and stoplogs in water passages. EM 1110-2-1602, Chapter 3, provides pertinent information on how to size and determine the locations of air vents.

6-6. Abrasion- and Cavitation-Resistant Surfaces

a. General. Consideration should be given to all water surfaces in the structure that may be subject to abrasion resulting from sediment flow and potential cavitation caused by high velocities and surface irregularities. All operating conditions, including diversion, should be evaluated for potential adverse effects.

b. Provisions for abrasion and cavitation protection. For concrete surfaces, the use of stainless steel linings, special concrete, abrasion-resistant coatings, and special concrete specifications to provide a high quality surface may be necessary to attain the required protection. Special concretes may include special aggregate concrete and silica fume concrete. Areas in the concrete specifications that should be addressed include the use of steel forms, additional forming support, restrictions on form joint locations, tighter surface tolerances, placement sequencing, prohibiting positive projections into the flow, and required submittals for critical concrete operations.

c. Repair. Specific methods for concrete repairs, patching, and grinding for construction deficiencies should be clearly defined in the specifications. Typical repair methods may not be satisfactory to provide the desired long-term durability. Repairs in high-velocity areas may require excavation to a depth that will allow bonding of the patch to the near-face reinforcement.

6-7. Corrosion Control

Gates, liners, piping, and other ferrous equipment and materials shall be protected against corrosion by coating and/or cathodic protection. For further information refer to UFGS 09965A, Painting: Hydraulic Structures; UFGS 09971A, Metalizing: Hydraulic Structures; TM 5-811-7, Electrical Design, Cathodic Protection; and the U.S. Army Construction Engineering Research Laboratory paint lab, Champaign, IL.

6-8. Operations and Maintenance Considerations

An important operation and maintenance consideration for a newly constructed intake structure is to minimize staffing requirements. To accomplish this objective, a remote control system and automated surveillance systems should be considered. To reduce maintenance, materials should be selected with extended service life in mind. The materials selected should resist corrosion and abrasion and minimize periodic maintenance. Features needed for maintenance should be provided including the maintenance bulkheads and bulkhead slots required for the on-site maintenance of emergency and service gates.

6-9. Instrumentation

Structural behavior instrumentation programs are provided for intake structures to measure the structural integrity, check design assumptions, and monitor the behavior of the foundation and structure during construction and the various operating phases. The extent of instrumentation at projects will vary depending on particular site conditions, the size of the structure, and needs for monitoring critical sections. Instrumentation can be grouped into those that either directly or indirectly measure conditions related to the safety of the structure. Plumbness, alignment, uplift, and seismic instruments fall into the category of safety instruments. In the other group, the instruments measure quantities such as stress and strain, length change, pore pressure, leakage, and temperature change. EM 1110-2-4300 provides details and guidance on the planning of instrumentation programs, types of instruments, and the preparation, installation, and collection of data.